Engineering News

Vol. 67, No. 17. 505 Pearl Street, New York. April 25, 1912.

An Old Iron Truss Bridge, 1853-1912

A relic of the early days of bridge construction in America was found and fully examined recently by the bridge engineers of the New York Central & Hudson River R.R. in the course of line-improvement work near Wakefield, in the northeasterly part of New York City. The bridge is a combination cast-iron and wrought-iron structure, and spans about 70 ft. It would today be classed as of the Pratt type, but it was built under a patent issued in 1839 to Col. Stephen H. Long. That the bridge has survived until the present day in the immediate vicinity of New York City is especially interesting.

The two photographic views, Figs. 1 and 2, herewith, were taken in the course of an examination of the bridge, and a report on it made a short time ago by the engineering department of the New York Central. As the pictures show, the structure consists of two parallel-chord trusses with cast-iron top chord and posts and wrought-iron bar diagonals and bottom chords. The floor was entirely of timber and the floorbeams were extended out beyond the trusses to form sidewalk cantilevers. The flooring of both roadway and sidewalk was of plank.

The bridge was exceedingly primitive in design, as regards both distribution of material and load-carrying capacity. The details show much ingenuity, and some of them are quite commendable, considering the means available at the time of building. On analysis, however, it is found that the different members are very unequally stressed. Further, the bridge is too weak to carry more than the very lightest highway traffic. According to the stress calculations made by the engineers of the New York Central (see tabulation below), a single heavy team with loaded wagon would stress many parts of the bridge above the working stresses usually allowed in present-day practice. The over-stressing is most marked in the wrought-iron members; the stresses in the cast-iron chords and posts were low. However, many of the cast-iron members are broken, as described more particularly below.

For a concise statement of the condition of the bridge and other circumstances concerning it, we quote from a special report made by C. P. Marsh, Bridge Engineer, to J. L. Holst, Engineer of Structures, Electric Zone, New York Central & Hudson River R.R., under date of Mar. 14, 1912:

LOCATION—This bridge was built to carry what is now DeMilt Ave. or 242d St., over the Bronx River, about 100 ft. west of the present southbound track of the N. Y. C. & H. R. R.R. The proposed change in alignment of track through Wakefield and Mt. Vernon making it necessary to shift the original course of the Bronx River at this location, the bridge is left spanning a mudhole.

DESCRIPTION—The bridge, consisting of a 14-ft. roadway and two 5-ft. sidewalks, was carried by two pin-connected trusses 69 ft. 9 in. long, which are designed after the principle of the Pratt truss with vertical end posts and counterbracing in each panel.

MATERIAL—The floorbeams, stringers and floor are of wood; top chord and posts of cast iron; other members of wrought iron. The bottom chord is composed of two 4x½-in. plain bars spliced with two 1-in. bolts, while the main diagonals and counters are forged eyebars.

The top chord is composed of a very good grade of cast iron, being of such a nature as to stand considerable mutilation before fracture.

All members except the diagonals (eyebars) are of uniform section throughout; the diagonals increase in section somewhat toward the end and the counter-diagonals decrease toward the end.

ADJUSTING WEDGES—The diagonals and counters are of fixed length, and the chords were evidently drilled for the pins before erection. Whatever slack there may have been was taken up by driving two opposed long wrought-iron wedges between the post and top chord section [see detail of post in Fig. 3 herewith].

PRESENT CONDITION—On account of the dangerous condition of the trusses, this bridge was closed to all but pedestrian traffic for some time previous to the beginning of the Mt. Vernon improvement.

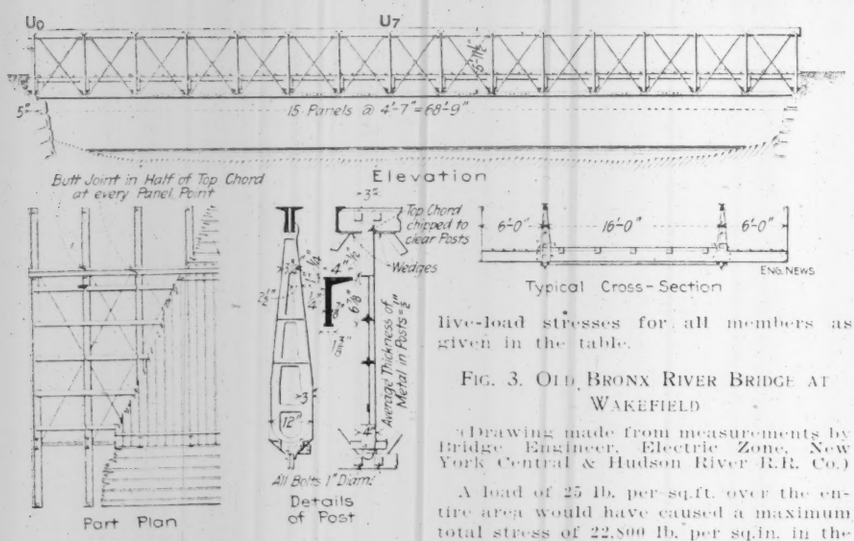
The westerly end post of the north truss is broken and the upper part of it gone [see Fig. 1]. The diagonally opposite end post is also cracked about midway between chords, while one of the prongs of the upper end which serve to hold the top chord in line is broken from a number of intermediate posts.

The recent use of this bridge was made possible only by placing wooden bents under it, converting it into a sort of trestle [see Fig. 2].

The top chord is in very poor alignment now, some portions deviating approximately 1 in. from a straight line joining the two ends. Half of it is spliced at every panel point, with no provision for transferring stress from one section to the adjoining one except by relying upon butt joints. Some of these have very poor contact, and at one point at least it was found necessary to resort to wedges to fill the space between the ends of the chord which projected beyond the diagonal pin hole, as they were too short to come to a bearing upon

FIGS. 1 AND 2. OLD IRON BRIDGE OVER BRONX RIVER AT WAKEFIELD, N. Y., BUILT 1853 UNDER LONG'S PATENT (1839)

(Cast-iron top chord and posts. Note broken end-post of near truss in left-hand view, and trestle support in right-hand view.)



each other. The ratio of the length of the top chord to its least radius of gyration is approximately 300. It should further be noted that the construction of the posts is such that it adds very little if any lateral stiffness to this chord.

The deviation noted above may have increased when the wooden bents were placed under the bridge; otherwise it seems remarkable that with one-half the chord entirely cut away and no support to the other half afforded by the posts the structure should have carried even the small live load which, according to the stress calculation noted farther on, it is able to carry.

CORROSION — Outward appearances indicate that the structure has suffered very little from corrosion. The material in the cast posts and top chord is in excellent condition; while the bottom chord and the diagonals have suffered more loss from pitting, still it is estimated that their cross-section has not decreased more than 10%. It is interesting to note that aside from accidental injuries the trusses are still in good condition, 60 years after construction, with very little or no maintenance.

HISTORY—The top chord bears the following name plates cast integrally with it: "Patented 1839 by S. H. Long." "Built 1853 by M. M. White." I find that Col. Stephen H. Long took out a patent in 1830 for a timber truss connected with wooden beams, and in 1836 published a 75-page pamphlet describing it.* In 1839 he took out another patent, a distinctive feature of which as set forth in the patent is as follows: "Posts to act in thrust and diagonals in tension, which is opposite that used in most construction." Nothing is said regarding distribution of stresses. I have been unable to find out anything about Mr. White.

ANALYSIS OF STRESSES—The dead-load and live-load stresses in pounds per square inch are given in the table below. The live-load adopted for the computation, a 4½-ton load, would hardly be considered sufficient for a highway bridge today. In arriving at this load consideration was given the dead-load stresses, which it was found were so high for certain members that only a small margin remained for live-load.

A load of 14 lb. per sq.ft. over the entire roadway and sidewalk would produce approximately the same maximum live-load stresses for all members as given in the table.

FIG. 3. OLD BRONX RIVER BRIDGE AT WAKEFIELD

(Drawing made from measurements by Bridge Engineer, Electric Zone, New York Central & Hudson River R.R. Co.)

A load of 25 lb. per sq.ft. over the entire area would have caused a maximum total stress of 22,800 lb. per sq.in. in the bottom chord.

RESULTS OF CALCULATIONS OF STRESSES IN BRONX RIVER BRIDGE

For loads assumed as follows: Dead load, including roadway floor, 3 in. thick, and sidewalk, 2 in. thick; live-load, two-horse team, 9000 lb., consisting of 6000-lb. wagon on axles 8½ ft. apart and 6-ft. gage, and horse load, 3000 lb., 9 ft. in front of forward axle.

Member	D L	L L	D + L
TOP CHORD (cast iron; sectional area, 16.5 sq.in.)			
U_0-U_1	+ 575	+ 270	+ 845
U_1-U_2	+ 1,070	+ 485	+ 1,555
U_2-U_3	+ 1,480	+ 660	+ 2,140
U_3-U_4	+ 1,810	+ 790	+ 2,600
U_4-U_5	+ 2,060	+ 885	+ 2,945
U_5-U_6	+ 2,220	+ 930	+ 3,150
U_6-U_7	+ 2,300	+ 930	+ 3,230
U_7-U_7	+ 2,300	+ 1,000	+ 3,300
BOTTOM CHORD (wrought iron; 2 bars, 4x½ in.; net area, 3 sq.in.)			
L_0-L_1	0	0	0
L_1-L_2	— 3,160	— 1,450	— 4,610
L_2-L_3	— 5,900	— 2,660	— 8,560
L_3-L_4	— 8,150	— 3,630	— 11,780
L_4-L_5	— 10,000	— 4,330	— 14,330
L_5-L_6	— 11,300	— 4,875	— 16,175
L_6-L_7	— 12,200	— 5,100	— 17,300
L_7-L_7	— 12,600	— 5,500	— 18,100
POSTS (cast iron; sectional area at throat near top, 4.25 sq.in.)			
U_0-L_0	+ 2,800	+ 1,280	+ 4,080
U_1-L_1	+ 2,400	+ 1,175	+ 3,575
U_2-L_2	+ 2,000	+ 1,070	+ 3,070
U_3-L_3	+ 1,600	+ 965	+ 2,565
U_4-L_4	+ 1,200	+ 860	+ 2,060
U_5-L_5	+ 800	+ 750	+ 1,550
U_6-L_6	+ 400	+ 650	+ 1,050
U_7-L_7	0	+ 540	+ 540
DIAGONALS (wrought iron; end panel, 1 eye-bar, 2x⅝; area 1.25 sq.in.; 3 panels, 2x½, 1 sq.in.; 4 panels, 1¾x½, 0.875 sq.in.)			
U_0-L_1	— 11,700	— 5,400	— 17,100
U_1-L_2	— 12,500	— 6,200	— 18,700
U_2-L_3	— 10,450	— 5,500	— 15,950
U_3-L_4	— 8,400	— 5,100	— 13,500
U_4-L_5	— 7,150	— 5,150	— 12,300
U_5-L_6	— 4,800	— 4,500	— 9,300
U_6-L_7	— 2,400	— 3,900	— 6350
U_7-L_7	0	— 3,260	— 3,260
COUNTERS (wrought iron; 3 end panels, 1 eyebar 1½x½, area 0.75 sq.in.; 5 panels, 1 eyebar, 1¾x½, area 0.875 sq.in.)			
L_0-U_1			
L_1-U_2			
L_2-U_3			
L_3-U_4			
L_4-U_5			
L_5-U_6	+ 4,800	— 1,950	+ 2,850
L_6-U_7	+ 2,400	— 2,620	— 220
L_7-U_7	0	— 3,260	— 3,260
UPPER BOLT OF END DIAGONAL:			
Shear			12,250
Bearing			34,500
BOLTS OF BOTTOM-CHORD SPLICE, near center of span:			
Shear			14,850
Bearing			26,000

As to whether the patentee or the designer of the bridge had some idea of the proper distribution of stresses, the internal evidence of the structure would indicate that his conception was correct as to what kind of stresses each member would be called upon to resist and in a general way as to their distribution. From the table of stresses it will be noted that the maximum diagonal stress occurs in the second panel from the end. This fact, and the uniform chord and post sections above-mentioned, point to the conclusion that the members were proportioned according to the best judgment of the builder and that probably the exact distribution of stresses was not known. However, Squire Whipple's work on bridge building, which contains the first correct and adequate analysis of a truss subjected to static and moving loads, was published in 1847, and might have been in the hands of the designer.

We are indebted to George A. Harwood, Chief Engineer, Electric Zone improvement, New York Central & Hudson River R.R., for the opportunity to make public the facts contained in this report.

Circular Reinforced-Concrete Reservoirs*

BY ALEXANDER POTTER†

The design of a circular reinforced-concrete reservoir appears to be so very simple that the inexperienced designer is apt to create a structure of larger diameter than the application of the simple formula of tank design would seem to warrant. To him there appears to be no ostensible reason why a structure twice the size of one already built should not offer every evidence of strength and stability if designed in accordance with the formula for ring tension—a very misleading deduction. The secondary stresses, which in small structures are insignificant, and consequently deemed of too little importance to have attention called to them, increase rapidly with the size of the structure, and only too often limit the size to which any particular type of construction can be adopted. In a circular reinforced-concrete tank, the writer has in mind the varying tension from point to point in the steel reinforcement due to the difficulty of obtaining a true circle in the field. In a small tank, this is not so serious, as its effect upon the resultant stresses in the steel reinforcement is so slight.

To make this point clear, some computations have been made by the writer based on the assumption that in the construction of this type, even with the best of care taken in the fields, a variation of ½ in. in the middle ordinate of a 10-ft. chord is likely to occur. These figures are given in the accompanying table.

*History of Bridge Engineering, by H. G. Tyrrell, page 138.

*Portion of a paper delivered before the New England Water Works Association, Boston, Mass., Feb. 14, 1912.

†Consulting and Contracting Engineer, 114 Liberty St., New York City.

TABLE SHOWING EFFECT OF INACCURACIES IN THE CIRCLE ON STEEL STRESS IN A CIRCULAR REINFORCED CONCRETE TANK

Radius of tank	Variation in radius of curvature due to a variation of ½-in. in middle ordinate of a 10-ft. chord	Corresponding range of tension in steel reinforcement. Average unit tension equals 14,000 lb. per sq.in.
25 ft.	23.0– 27.2	12,860 lb.–16,200 lb.
50 ft.	42.8– 60.1	11,950 lb.–19,100 lb.
100 ft.	74.9–150.5	5,910 lb.–21,100 lb.
200 ft.	120.2–632.0	5,820 lb.–14,500 lb.

This table is not made to show accurately the variation in the tensile stresses of the steel reinforcement for the various diameters given; it does, however, give a fair idea of what may be expected in the variation of the ring tension in a circular structure. It points out the danger resulting from carelessness in constructing a circular reservoir more than 100 ft. in diameter. For reservoirs of large diameter, however, the economy resulting from the use of a circular section does not obtain to the same extent and consequently recourse to this type is not so frequent.

The writer's experience would tend to limit the working tension in the steel reinforcement to 14,000 lb. per sq.in. in a small tank, and to 12,000 lb. per sq.in. for comparatively large tanks. A reduction in the allowable steel tension for large tanks is recommended because of the greater range in the ring tension present in the larger structure. It may even be advisable to reduce the allowable unit stresses below 12,000 lb. per sq.in. to keep within safe limits the excessive local stresses which cannot be avoided.

The variation in the tension of the steel reinforcement from point to point due to the varying curvature of the shell, makes the use of a reinforcing bar with mechanical bond advisable. The reinforcing bars for this reason should also be of as small a size as it is possible to handle economically in the field. A high-carbon steel with an elastic limit of 50,000 lb. per sq.in. can be used to great advantage.

Another difficulty to be considered in the design of a circular reservoir is the tendency to rupture along the line between the inside wall of the reservoir and the base, due to the expansion of the walls by internal water pressure and the consequent drawing away, as it were, from the base of the tank.

One of the greatest advantages possessed by the circular section and not possessed by any other, is the ability to increase economically the capacity of such a reservoir by simply increasing its depth. The ability thus to increase the capacity of a reservoir is of great importance in the design of water-works improvements. It enables the designer to keep down the first cost of construction by building a reservoir of a size sufficient for the immediate needs. As the water consumption increases, it is possible to increase economically the capacity of the reservoir, and at the same time raise the water level to counteract the increasing frictional losses in the distribution system due to increased consumption.

A good example of increasing the capacity of a circular reservoir is the enlargement of the distribution reservoir for the village of Suffern, N. Y. This village takes its water supply from Anthrim Lake, formed by impounding a branch of the Ramapo River. The water is pumped from this lake to a distribution reservoir located on the side of a mountain to the north of the village, about 180 ft. above the average village datum. This distribution reservoir, built a number of years ago, is a circular tank 70 ft. in diameter and 10 ft. 6 in. deep, sunk entirely into the ground. The walls forming the sides of the tank are 2 ft. thick, and both bottom and sides are constructed of plain concrete. This reservoir has a storage capacity of 266,000 gal. and cost approximately $4000.

The recent growth of the village has made it advisable to double the capacity of this distribution reservoir. The old structure, although massive, nevertheless leaked to a considerable extent, especially in the bottom. It was, therefore, decided to line the bottom of the tank at the same time that the sides were being raised. The reservoir as remodeled has an inside diameter of 69 ft. and holds approximately 20 ft. of water, giving a storage of 659,000 gal. The side walls of the old tank are lined on the inside with 6 in. of reinforced concrete. Above the old work the width of the new work is 12 in. tapering to 8 in. at the top.

The circumferential reinforcement consists of ⅝-in. square corrugated bars, possessing an elastic limit of 50,000 lb. per sq.in. These bars are so spaced that the average unit tensile stress in them does not exceed 14,000 lb. per sq.in.

In designing the lining for the old reservoir, it was assumed that the reinforcement would only have to take care of the increased tension due to the additional depth of 10 ft. 6 in. This is a constant quantity with a full reservoir; consequently the spacing and size of the steel in the lining of the old reservoir are uniform. The existing wall is still relied upon to resist the hydrostatic pressure that it formerly did.

It is not very likely, because of the great daily fluctuation in the water level in this reservoir, that ice pressure will develop to such an extent as to overstress seriously the reinforced-concrete shell, and consequently, no provision has been made for such pressures.

The bottom lining is reinforced with ⅝-in. square corrugated bars, which have their ends hooked over the lowest reinforcing ring. Vertical ⅝-in. square bars, spaced 3 ft. centers, were used, as vertical distributors. Each reinforcing ring is made up of six sections lapped 30 in. and wired. The rings were also wired to the vertical reinforcement at every intersection.

The thickness of the bottom lining varies from 3 to 6 in., so arranged as to offer better drainage than was obtained in the old tank.

The reservoir was completed on Oct. 12, 1911, and filled for the first time to its full depth, on Nov. 11, 1911. No leaks whatever have thus far appeared. The only precaution to render the reservoir watertight other than that of using a fairly wet concrete which was mixed in the proportion of one part of cement to two parts of sand and four parts of ¾-in. broken trap rock was to wash the inside of the tank with a semi-liquid cement.

The writer wishes to call attention to the comparatively low cost of this work. An increase in the storage capacity of 294,000 gal. was obtained at a cost of $2500, the contract price for this work. The location of the reservoir on a steep mountain slope about 180 ft. above the street level, added considerably to the cost of hauling the structural material to the site, and consequently, to the contract price.

A New Process of Coating Iron and steel with alloys consisting chiefly of lead or zinc has been devised and patented by Prof. Herman J. Lohmann, of New York City, and is known as "Lohmannizing." With this process the metal is first dipped in a bath of dilute sulphuric acid, as in the galvanizing process, to remove rust, scale and dirt, and then into a solution composed of an acid and an amalgamating salt, a patent compound known as a "Lohmann bath," which further cleanses the pores and cavities. After this bath the metal is dried and immersed in a molten alloy. It is claimed that an amalgam or chemical union is thus formed between the amalgamating salt and the protective alloy. The treated metal is then raised in temperature by the heat of the molten alloy, which is at a temperature of 950° to 1000° F. When the metal reaches a temperature of 500°, it is claimed that the amalgamating element volatilizes, is released and passes off from the surface of the alloy, thus freeing the surface of the metal, including pores and cavities, from all oxidizing agents, and allowing the alloy to fill every tiny pore and crevice. The oxygen present in the pores, having greater affinity for the amalgam element than for either the base metal or alloy, passes into the atmosphere and thus removes a frequent source of failure in galvanized metal. It is claimed that the long slender tentacles of alloy penetrating into the pores of the base metal give the protective coating a much better hold than in the ordinary galvanizing process. The proof of this penetration of the alloy is established by micrographs of sections of a coated sheet. As previously noted, the process is used for coatings of both zinc and lead alloys. The Brier Hill Steel Co., Youngstown, Ohio, are licensees for the use of this process, which is owned by Prof. Lohmann.

The New Kensico Dam

By Alfred D. Flinn *

The Kensico reservoir will be the main distributing center for the 500,000,000 gal. per day water supply for the city of New York, now under construction. At this site, about 15 miles north of the northern boundaries of the city, a storage of forty billion gallons will be made behind the mammoth masonry dam, described in this article.

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About fifteen miles beyond New York City's northerly boundary was Lake Kensico, a small reservoir developed about thirty years ago as a part of the water supply of the Borough of the Bronx. This reservoir lay in the bottom of a large basin which is the only available site near the city for a great storage and distribution reservoir along the Catskill aqueduct. Here a capacity of approximately 40,000,000,000 gal. can be utilized by the construction of a great masonry dam. Kensico reservoir will be sufficient to supply the city for several weeks without receiving water from Catskill impounding reservoirs, thus permitting the portion of the aqueduct to the north to be cleaned or repaired. In the future it will also become the center from which other main delivery conduits will radiate into the metropolitan district.

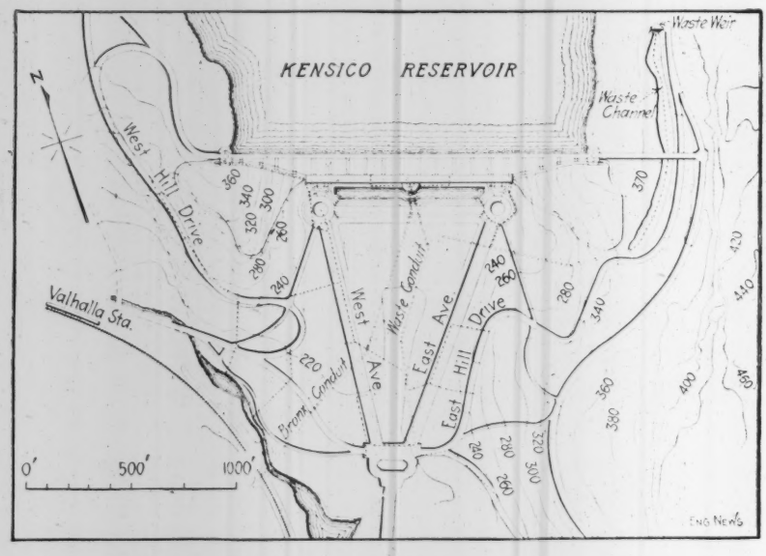


FIG. 1. GENERAL PLAN OF KENSICO DAM AND GROUNDS
(Old dam now located about 200 ft. down stream from new.)

Kensico dam will be 1830 ft. long on top and its expected maximum height is 300 ft. above the deepest part of the excavation. The top of the dam will be 170 ft. above the old stream bed, but only 130 ft. above the surface to which the earth will be graded at the completion of the work; it will be 15 ft. above water level when the reservoir is just full. The dam's minimum thickness at the top will be about 28 ft. and its maximum thickness at the bottom about 230 ft. (Figs. 1 and 2). Approximately 1,000,000 cu.yd. of masonry are required and the excavations will total a million and a quarter yards. Of the masonry the great bulk will be cyclopean, with the concrete mixed approximately 1:3:6, using local sand and gneissoid granite.

On its upstream side the dam will be faced with large, dense, cast concrete blocks, similar to those used on the Olive Bridge dam, of the Ashokan reservoir, farther north on the same water supply. Below ground the downstream face will be cast against forms in place, while the exposed portions will be of granite. Owing to its very conspicuous location, the dam is to receive architectural treatment which, while simple and dignified, is intended to be commensurate with the magnitude and importance of the structure. Recognizing the advantages of having an architect's assistance, a design for the downstream face and its connected structures was obtained from Messrs. York & Sawyer, of New York. To dispose of the vast quantity of excavated material pleasingly and create a suitable foreground for the dam a landscape design was obtained from Charles W. Leavitt, Jr., of New York. Figs. 1 and 4 show the general appearance of the dam and its surroundings, as proposed.

On Dec. 21, 1909, four bids were received for the contract, known as Contract No. 9. The lowest bid amounted to $7,953,050, and was submitted by John C. Rodgers, James M. Rodgers and John J. Hagerty, to whom the contract was awarded. The average of all bids was $8,888,775.

Various schemes for a large reservoir in this basin, or in part of it, had been proposed at several times. When the final project was first suggested, a large dam just below the old dam was intended Very extensive borings, as well as topographic surveys, demonstrated that the best site for a high masonry dam was a few hundred feet upstream from the old dam. But this site could not be utilized unless substitute water storage could be provided for the period of construction. However, it was found readily feasible to construct two small earth dams in branches of the main valley, well below the future flow level, and thus to create two reservoirs of much greater capacity than the old one. Careful esimates of cost showed that even when the expense of the substitute temporary reservoirs and pipe line were included, a very large saving could be effected by building the dam on the upper site, which is the one shown in the plan in Fig. 1.

Lake Kensico had a maximum depth just back of the old dam of 45 ft.; the new dam will raise the water level 110 ft., or to elevation 355 above tide. The new reservoir will have a water surface of 3.75 sq.mi. and a very irregular shore line about 40 mi. long. New York City has acquired 4481 acres altogether, so that there will be a park fringe of no mean proportions surrounding the new reservoir and one very large island within it. The flooding of the valley has necessitated the building of 8.5 mi. of first-class highways and the construction of three bridges. For economical as well as esthetic reasons, reinforced concrete was

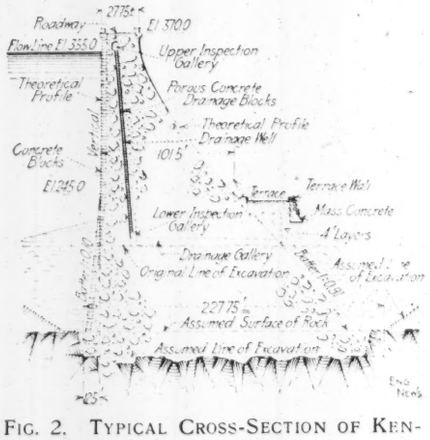


FIG. 2. TYPICAL CROSS-SECTION OF KENSICO DAM

the material chosen for all three bridges. Two are girders, Bear Gutter bridge having three 38.5-ft. spans and Cranberry Brook bridge five exactly similar spans. Rye bridge has five segmental arches of about 135-ft. span. All three bridges are nearing completion. Fig. 3 is a view of the dam site with the old dam shown to the left and the line of the new dam directly in front of the camera.

Contract No. 9 includes all the works mentioned. For their construction 10 years was allowed, the first two being allotted to the preliminary operations. Work was begun Feb. 14, 1909, the substitute reservoirs were ready some time before September, 1911, and the old reservoir was entirely drained between Sept. 9 and Sept. 29, 1911, so that excavation of the main foundation pit could be begun. Meanwhile the contract had been assigned, and is now in the name of H. S. Kerbaugh, Inc., with Beverly R. Value as general superintendent and engineer, and B. C. Collier as resident engineer.

Borings and test pits, aggregating 75 in number and 3045 ft. in depth were made along various lines at the site of the dam. Fig. 5 is a profile on the dam line showing the borings there.

FIG. 3. VIEW OF OLD LAKE KENSICO AFTER DRAINAGE

Old dam at left. Line of new dam marked by white post in foreground and track on opposite hill.

FOUNDATIONS

At Kensico dam site the rock formations are quite similar to those at New Croton dam. The axis of the dam lies approximately east and west (S. 72° 02′ E.), with the downstream face southerly. On the easterly side of the valley a sound gneissoid granite merging into gneiss is found; on the westerly side the rock is a reasonably good schist; between these two lies a bed of Inwood limestone, some parts of which are almost a marble. Along the axis of the dam the limestone occupies a distance of 600 ft.

DESIGN

In designing the cross-section of the dam the following bases for computation were adopted: Weight of masonry per cubic foot, 146 lb.; maximum allowable pressure per square foot on masonry at downstream face, 15 tons; at upstream face, 20 tons; coefficient of friction, masonry on masonry, 0.67; masonry on rock, 0.67; ice pressure at full reservoir level (elevation of spillway, i.e., 15 ft. below top of dam), 23.5 tons per lin.ft. of dam; upward water pressure under dam, or in any joint, considered as base

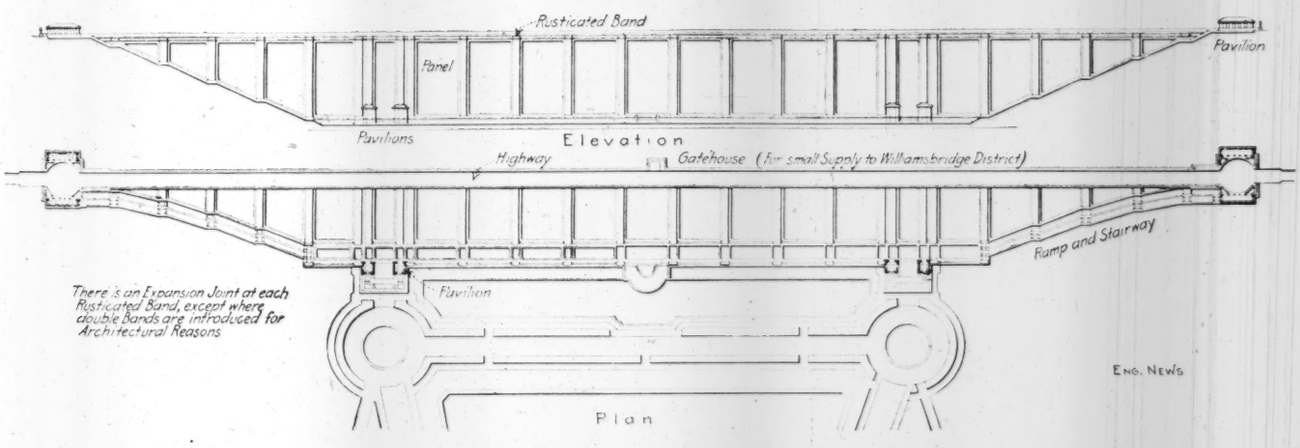


FIG. 4. PROPOSED ARCHITECTURAL TREATMENT OF KENSICO DAM

of portion above, equal to two-thirds the hydrostatic head at heel or upstream face, reducing uniformly to zero at toe, or downstream face; extreme flood level, 5 ft. above full reservoir level; resultant of thrusts must lie within middle third under all assumptions; tensile strength of masonry not directly taken into account in computations.

The unit weight of masonry used in the computations, 2⅓ times the weight of water, is much less than the actual, but closely agrees with the average total density of the dam, allowing for the galleries, wells and other void spaces within the mass of the dam.

Mathematical investigations of stability were made for following assumptions: (1) Reservoir with water 5 ft. above spillway, i.e., 10 ft. below top of dam, and upward water pressure; (2) water at full reservoir level, ice thrust and upward water pressure. Other combinations of forces and conditions were also considered, including ground water and earth on the toe of the dam.

Kensico dam was designed for a factor of safety of two under the most unfavorable condition, which obtained with water at normal full reservoir level, ice thrust and upward water thrust. Keeping the factor of safety against overturning about the toe at two made all stresses so moderate and the coefficient of friction so reasonable that the limits mentioned in a preceding paragraph were approached only in the highest part of the dam. Here the pressure at the upstream face, at the base, with reservoir empty, was computed at about 19 tons per sq.ft. (but this condition is improbable of realization on account of requiring water to be stored in the reservoir before the completion of the dam); and the pressure on the downstream face may possibly go as high as 14 tons per sq.ft., but probably only for a length of relatively a few feet, the height of section decreasing rapidly in both directions. The cross-section is so liberal that the dam will have abundant stability even if a surface layer of the masonry is rendered partially useless in resisting compressive stresses through temperature action or deterioration through long-continued exposure. Besides, the portions of the dam in which the highest stress intensities exist are to be thoroughly protected by earth.

ANALYTICAL COMPUTATIONS.—For the analytical computations the following formulas, with modifications for special cases, were employed:

Using symbols and units shown in the diagram in Fig. 6, with the following additional premises:

Lengths in feet.

Weight of masonry is taken as 2⅓ times weight of water = (146 lb. per cu.ft.).

Horizontal plane joints assumed at various convenient elevations, or distances below top of dam, for purposes of computation.

Moments of forces taken about intersection of line of action of W with joint, the forces being represented by their resultants and considered to act in the same vertical plane.

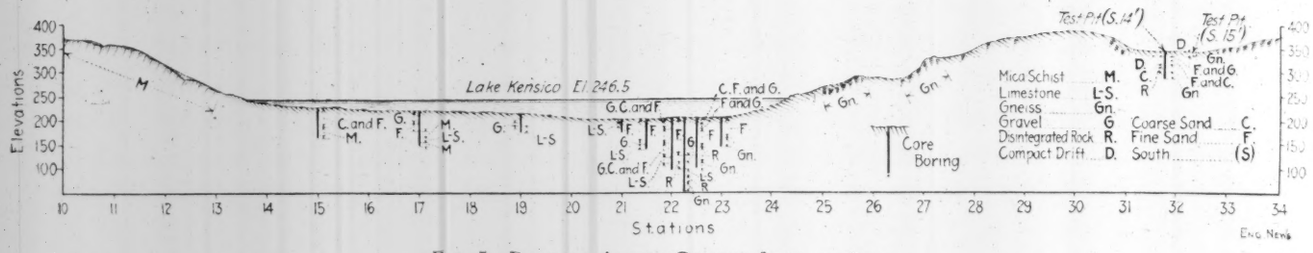
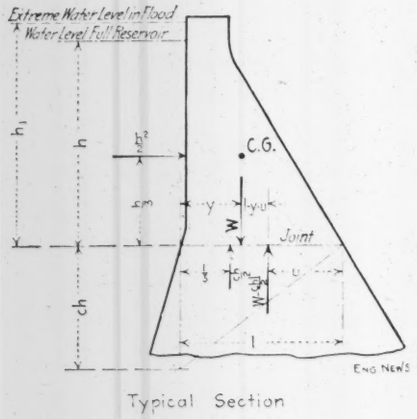


FIG. 5. BORINGS ALONG CENTER LINE OF DAM



Typical Section

Symbols and Units:

W = Weight of Masonry above Joint

T = Thrust of Ice at Water Surface, considered Horizontal

h^2 = Horizontal Thrust due to Static Pressure of Water against the Upstream Face of the Dam

h_1^2 = Ditto for Water at Flood Level

ch = Intensity of Pressure of Water vertically upward at Upstream side of Joint, it is considered to decrease uniformly to Zero at the Downstream Side

Unit of Weights and Forces is the Weight of One Cubic Foot of Water (62.5 lb.)

FIG. 6

A cross-sectional slice of the dam 1 ft. thick is used for computation and so the forces are those acting upon 1 lin.ft. of the dam.

Begin with assumed profiles and modify, if necessary.

The following formulas determine the position, under various assumptions of the intersection of the resultant of the forces acting on the dam with the joint in question. In other words they determine the position of the "line of pressure."

GENERAL EQUATIONS FOR STABILITY AGAINST OVERTURNING

$$Th + \frac{h^3}{6} + \frac{chly}{2} - \frac{chl^2}{6} - (l - y - u)\left(W - \frac{chl}{2}\right) = 0$$

1st Case—Water pressure only, water at full reservoir level. $T = 0$. $C = 0$.

$$l - y - u = \frac{h^3}{6W} \text{ or } \frac{\frac{h^2}{W}}{6\frac{W}{h}}$$

2d Case—Water pressure and ice pressure, water at full reservoir level. $C = 0$.

$$l - y - u = \frac{h}{6W}(6T + h^2) \text{ or } \frac{6T + h^2}{6\frac{W}{h}}$$

3d Case—Water pressure and upward pressure beneath joint, water at full reservoir level. $T = 0$.

$$l - y - u = \frac{h^2 + cl(3y - l)}{6\frac{W}{h} - 3cl}$$

For water at flood level substitute h_1 for h.

4th Case—Water pressure, ice pressure and upward pressure.

$$l - y - u = \frac{6T + h^2 + cl(3y - l)}{6\frac{W}{h} - 3cl}$$

Modifications of these formulas and additional formulas were devised by Designing Engineer C. E. Gregory and Assistant Designing Engineers W. M. Smith, Sr., and O. L. Brodie. A statement of these can be found in the *Proceedings* of the American Society of Civil Engineers, March, 1912, and in "Masonry Dams," by Morrison and Brodie.

As an independent check graphical methods were used. Constructing a theoretical profile by mathematical processes was not, however, the only method employed. Having a large collection of the cross-sections of masonry dams, as guides, it was more direct and simpler to draw a trial section, test it analytically according to the conditions for the case in hand and modify this section two or three times. Results of the final computations are shown in Fig. 7.

PROFILE ADOPTED—Fig. 8 shows the cross-section adopted, in comparison with several well known high masonry dams, straight in plan. It is, and is intended to be, unquestionably and conservatively safe, for besides the great value to the city of the stored water, the valley of the Bronx River below the dam is heavily populated, the river flowing through several large towns and the Borough of the Bronx, New York City.

Stability and security additional to that shown by theoretical computations are to be obtained through other features of the design and the methods of construction. Notwithstanding that the dam has been designed of ample dimensions to resist successfully the worst conceivable conditions, in its construction every effort will be made to obviate undesirable conditions. The foundations are to be carried down well into sound rock and the masonry so keyed into the rock that sliding will be impossible at the base. Vertical bonding of the cyclopean masonry, and thoroughgoing precautions to prevent horizontal joints in the concrete, such as the removal of laitance and "dead" mortar, will prevent sliding on any surface within the masonry.

EXPANSION JOINTS—Wide temperature ranges are inevitable in large masses of masonry, built rapidly with portland cement. Contraction, with accompanying cracks, must follow, unless controlled. After observations on a number of masonry dams of various ages, it was decided to provide expansion (or contraction) joints in the masonry dams of the Catskill water-works at intervals of approximately 80 ft. In Kensico dam the expansion joints are 75 ft. apart in the central portion, and 77 and 79 ft. in the hillside parts; they do not extend directly through the dam, but are tongued and grooved as shown in Fig. 10; all faces of each joint are vertical and lubricated to prevent adhesion of fresh concrete to that previously deposited. Near the upstream end of each expansion joint an inspection well is provided. After the temperature changes shall have settled down to their normal seasonal range, further resistance to the passage of water will be interposed by building into the masonry just upstream from the inspection wells the copper water-stops, as shown in larger detail by Fig. 10. Division of the dam into sections by the expansion joints permits building up one portion somewhat in advance of those adjacent, without recourse to racks in the masonry. But this practice cannot be permitted without restriction, for when a large new mass of masonry is keyed to another mass which has had time to cool and contract, stresses may be caused in the new mass as it contracts which may lead to cracks parallel to the axis of the dam.

DRAINAGE SYSTEM—Very few masonry dams have proved wholly impervious to water after being in service a short time. In the Olive Bridge dam and a few others of which the writer has knowledge, drainage systems have been constructed near the upstream face. The method used in the Olive Bridge dam was described and illustrated in ENGINEERING NEWS, Aug. 1, 1907, p. 112; that in the Kensico dam will be similar, consisting of porous concrete blocks with 16-in. diameter nearly vertical holes built up in the masonry so as to form continuous passages from an upper to a lower gallery (shown in Fig. 9). These passages, or wells, are 12 ft. apart, 10 ft. from the upstream face of the dam at their tops and 30 ft. at their bottoms. Seepage, if any, will be collected in a gutter in the lower gallery and discharged through a transverse drain leading out through the downstream face of the dam.

WASTE WEIR—The area of the reservoir is so great with respect to its natural watershed and the water entering by way of Catskill aqueduct is under such thorough control that a waste weir was scarcely necessary. However, as a conservative precaution, a waste weir 50 ft. long was provided at a convenient gully near the easterly end of the dam. (See plan in Fig. 1.) A reinforced-concrete conduit leads the overflowing water, if any, to the river channel. In the valley bottom, this conduit is connected with a 60-in. waste conduit through the lower part of the dam. For the gates governing the flow into this waste pipe and into the Bronx conduit, supplying a portion of the city, small gate chambers are to be formed in the dam. Pipes from these gate chambers will also supply fountains below the dam.

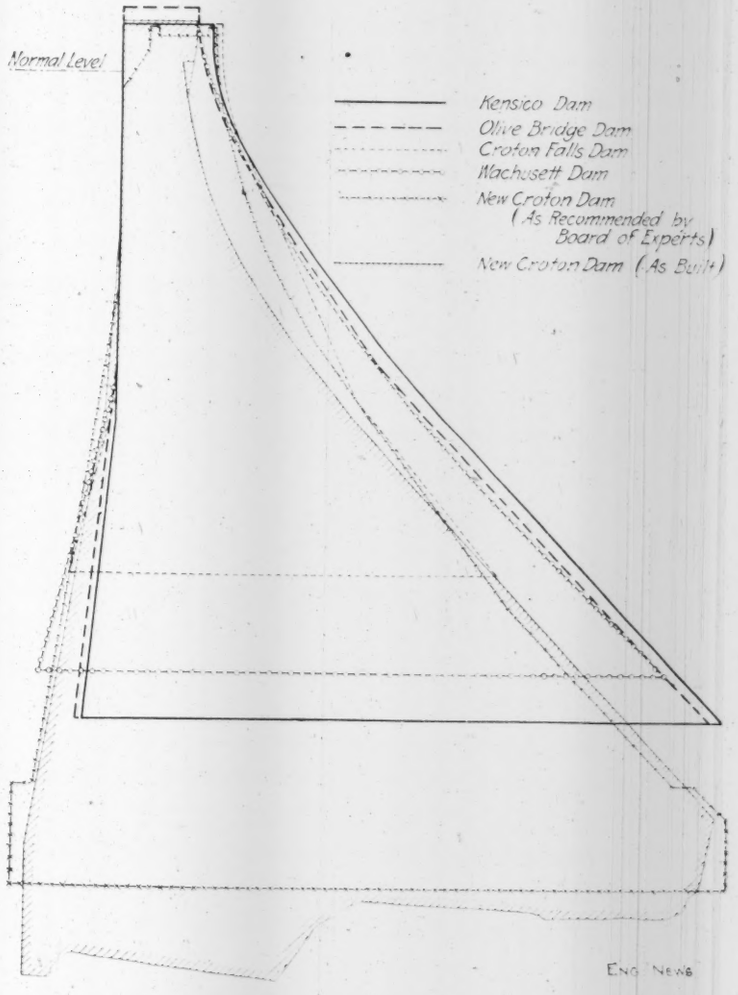


FIG. 7. COMPARATIVE PROFILES OF KENSICO AND OTHER WELL-KNOWN MASONRY DAMS

SPECIFICATIONS

Naturally the specifications for the Kensico dam were written with great care, keeping in mind the long term of the contract and providing for considerable variation in the architectural design, which had not been finally determined at the date of advertising. In addition the usual uncertainties and exigencies of a great masonry dam had to be covered. There are many interesting passages, some of which are quoted below. The general arrangement of the specification sections, based on the items for payment, and the scheme of numbering for con-

venience of reference are stated in the introductory note, which reads as follows:

Note—In numbering the sections of the specifications, the decimal system is used, the figure before the decimal point indicating the item number, and the figure after the decimal point the serial number of the section under the particular item. Where several items are grouped together, as Items 2, 3, 4, 5 and 6, the number of the first item of the group is placed before the decimal point, as 2.1, 2.2, 2.3, etc. The general sections have no decimal point. The sections relating to general requirements for a particular group of items have been given the number of the item directly before which each group is placed, together with the letter G, to distinguish them from the sections of that item, as 15G.1, 15G.2, etc., for masonry, and 16G.1, 16G.2, etc., for concrete.

[Inasmuch as this decimal classification has no significance unless the whole specification is before the readers, we have omitted the decimal designations from the following extracts from the specifications. Restrictions of column width and of typography also prevent an exact reproduction of the form of the specifications, so the portions given should be read only for their contents and not for their form.—ED. ENG. NEWS.]

For quick finding the appropriate item number is printed in large heavy figures on the upper outer corner of the page. At the bottom of each page the contract number, C—9, appears, so that if leaves be cut out, as for making up other contracts, they may be readily identified. These methods have been used in the Catskill contracts from an early date.

Perhaps one of the most novel features is the chapter on cut-stone masonry. In this an endeavor was made to reduce to orderly statement in the form of specifications the rules and practices of the stone-cutter's union, so far as these could be learned. Especial attention was given to allowances for irregular shaped stones and for the cutting of moldings, and for similar special work, the purpose being to provide items and methods for measurement and payment which could be applied to a considerable variety of shapes and surface cutting for which exact drawings were not available when the contract was drawn.

GENERAL SECTIONS

ROCK EXCAVATION—Rock shall be excavated for the masonry portion of the dam to a sufficient depth to secure a foundation on sound ledge-rock, free from open seams or other objectionable defects. A cutoff trench may be required under the whole or a part of the length of the dam. It is the intention to build the masonry against the sides of these rock excavations. To preserve these sides in the soundest possible condition, and to obtain over the whole foundation a rock surface free from open seams or cracks, unusual precautions will be required in excavating. Rock for the foundation may be removed by blasting, to the extent directed, with explosives of such power and in such quantities and positions as will neither crack nor damage the rock upon or against which the masonry is to be built, and whenever, in the opinion of the Engineer, further blasting is liable to injure the rock upon or against which masonry may be built, blasting shall be discontinued, and the excavating of the rock continued by wedging and barring, or other approved methods. Whenever directed, the method of excavating shall be changed from blasting to wedging and barring, or from wedging and barring to blasting. In order not to disturb the rock along the sides of the cutoff trench, and to maintain these sides as nearly vertical as practicable, channeling or other acceptable methods in connection with which light charges of explosives may be used to the extent permitted, shall be used.

PREPARATION FOR INSPECTION OF ROCK EXCAVATION—Whenever directed during the progress of rock excavation, all dirt and rock which shall have been loosened shall be removed from designated areas, and the surface of the rock shall be cleaned, using steam to melt the snow and ice, if necessary. The water shall subsequently be removed from the depressions so that the whole surface of the areas designated can be inspected to determine whether seams or other defects exist.

PREPARATION OF ROCK FOUNDATIONS—The surfaces of rock foundations shall be left sufficiently rough to bond well with the masonry, and, if required, shall be cut to rough benches or steps. Before any masonry is built on or against the rock, the latter shall be scrupulously freed from all dirt, gravel, boulders, scale, loose fragments, and other objectionable substances. Streams of water under sufficient pressure, stiff brushes, hammers and other effective means shall be used to accomplish this cleaning. Steam-jets shall be used to remove thoroughly ice or snow, if any be found on the rock when it is desired to lay masonry.

SPECIAL PREPARATION OF ROCK SURFACES

WORK INCLUDED—The foundation of the dam, not including any appurtenant structure, below Elev. 350, shall receive special preparation before masonry is built thereon. Other rock surfaces where water-tightness is desired, may be designated for similar special treatment.

DESCRIPTION OF WORK—Rock surfaces designated for special preparation shall be cleaned as required. Seams and cavities shall be traced as far as directed by drilling holes as required, or by other approved means. All such seams and cavities shall then be carefully cleaned out and filled with concrete, mortar, grout or clay. Wherever directed grout shall be pumped under such pressures as required up to 100 lb. per sq.in., through hose or pipe of at least 2 in. inside diameter, the connection between the hose or pipe and the rock being made tight. Portions or the whole of the designated area shall be plastered with rich mortar, if directed. This work, however unusual or tedious, shall be done very thoroughly.

MEASUREMENT AND PAYMENT—Each area of rock surface specially prepared in accordance with the above section shall be measured for payment as the number of square yards in the plan or horizontal projection of the whole area designated in the order, except that for areas so prepared on the sides of the cutoff trench or a corewall trench, the projections on a vertical plane through the axis of the trench shall be measured as well as the horizontal projections. Each such area shall be paid for but once, regardless of how many times it may have to be gone over in order to leave it in a satisfactory condition when the masonry is laid. For the special preparation of rock surfaces the price stipulated will be paid, which shall cover all costs incidental to such work, except the drilling of holes, cement and any grount or clay forced in under pressure. No area designated in any order shall be measured as less than 10 sq.yd.

MASONRY ITEMS

GENERAL CONDITIONS AND REQUIREMENTS

DESCRIPTION OF MASONRY—The greater part of the dam will be built of cyclopean masonry consisting of concrete in which stones of various sizes up to the largest that can be conveniently handled will be embedded. Concrete blocks will be used for facing portions of the dam and for other parts of the work. Stone will be used for facing the exposed portions of the down-stream side of the dam, and for structures connected with the dam. Mass concrete will be used for the waste-weir, core-walls, gate-chambers, retaining-walls, bridges, culverts and other structures or parts of structures shown on the drawings or otherwise required. Reinforced concrete will be used in parts of bridges, gate-chambers, conduits and other structures.

PREPARING FOUNDATIONS FOR MASONRY—Before laying masonry of any class at a given place, the rock or other foundation, or the masonry previously laid shall be properly prepared as specified or directed, and, except for dry rubble masonry and paving, a bed of fresh mortar of the thickness required shall be spread over the foundation and thoroughly worked into all depressions and crevices. No additional payment shall be made for this, it being included in the measurement of the masonry in connection with which it is used.

BUILDING FRESH MASONRY ON OLD—For the proper bonding of fresh masonry with that which has set or partially set, such provision shall be made of steps, dovetails or other devices or methods as may be prescribed. Each morning, or whenever fresh and old masonry are joined, the contact surfaces of the old masonry shall be thoroughly cleaned, using a stiff brush or other tools if required, and a stream of water under pressure, and it shall be clean and wet, but free from pools of water, at the moment the fresh masonry is placed. Special efforts shall be made to remove very thoroughly all laitance, waste mortar and other substances which would prevent complete cohesion of the masonry throughout the main body of the dam.

IMPERMEABLE MASONRY—Special efforts shall be made to render all classes of masonry in the dam, except concrete drainage blocks, as dense and impermeable as possible. All joints, except expansion joints, shall be thoroughly filled with mortar, and in facing masonry they shall be finished flush with the neat face as the masonry is laid, unless recessed joints are ordered. Particular attention shall be given to the up-stream face joints of the dam in order to obtain water-tight work. To this end detailed directions as to the preparation and working of the mortar shall be carefully followed. Raking out and repointing the joints shall be done only when specially directed. If at any time before the final completion of the work any face joint be found unsatisfactory, it shall be raked out to a depth of at least 2 in. and pointed, when and as directed, at the expense of the Contractor.

MASONRY TO BE KEPT MOIST—Every precaution shall be taken to prevent concrete from drying, until it shall have become so thoroughly set and hardened that there can be no danger of cracking from lack of moisture. Concrete or other masonry, if required, shall be kept moist for at least two weeks or until covered with earth, unless otherwise specified or directed.

INSPECTION WELLS—In connection with the expansion joints in the dam, inspection wells shall be constructed as shown on the drawings. These wells will not be filled under this contract, but may be filled with masonry or other material by the City before the completion of the work under this contract.

COPPER STRIPS IN EXPANSION JOINTS OF DAM—A strip of copper shall be placed across each expansion joint of the dam at the inspection wells, as shown on the drawings, to cut off leakage. A portion of the strip shall be placed in a groove in the vertical face forming one side of the expansion joint, and surrounded with concrete or mortar, allowing the remainder of the strip to project. After the concrete or mortar in the groove has set, parts of the projecting strip and portions of the vertical face of the masonry shall be coated with hot paraffin or other acceptable substance to prevent the adhesion of the strip to the concrete where it crosses the expansion joint. Concrete shall then be placed and carefully rammed around the projecting strip on the other side of the joint, care being taken to clean thoroughly the uncoated portions of the strip before the placing of concrete. Special care shall be taken to keep the concrete or mortar surrounding the strip, moist for such time as may be required.

STONE FOR MASONRY

STONES TO BE CLEANED—Stones for masonry wherever used shall be thoroughly cleaned before being brought to the place where they are to be used, by washing with water under pressure from a nozzle, by the use of brushes, or as otherwise directed, and shall be satisfactorily clean when placed in the masonry. Hammers or other tools shall be used, if necessary, to remove objectionable materials adhering to any stone. Stones shall be wet at the time of placing in the work.

STONES FOR CYCLOPEAN MASONRY—Stones to be used in cyclopean masonry shall be sound, clean, strong, insoluble and durable, of irregular shapes, but without long thin projections, and of various sizes, and shall be as large as can be economically quarried, transported and handled.

CYCLOPEAN MASONRY

WORK INCLUDED—Cyclopean masonry will be used for the greater part of the body of the dam, and shall be made of stones embedded in concrete made approximately of one volume of cement and nine volumes of aggregates. There shall be as great a proportion as possible, consistent with good work, of the largest stones that can be economically handled.

PLACING—The stone and concrete block facing of the dam, where used, shall be laid in advance of the cyclopean masonry so as to serve as forms for the latter, but shall not be built to a greater height than is authorized. Wet concrete in sufficient quantities shall be deposited in low places in the work, and large stones shall be lowered into it immediately as close together as possible. The concrete shall be carried to the work in bottom-dumping buckets, or as otherwise directed. The stones shall be joggled with a bar and otherwise jarred so as to settle well into the concrete, and the concrete shall be worked with suitable tools so as to force the escape of entrained air or surplus water, and assure the filling with concrete of all spaces around the large stones. In placing large stones, care shall be taken to secure as good bonding, both horizontally and vertically, as may be practicable with this class of masonry. Wherever possible, smaller stones shall be embedded in the concrete between the large stones. The object is to obtain a monolithic mass of stone and concrete, containing as large a proportion of stone and as impervious to water as it is possible to secure.

DIMENSION STONES

WORK INCLUDED—Dimension stones will be used for portions of the architectural features of the dam and appurtenant structures. A dimension stone shall be one having specific form and more than one required exact dimension.

JOINTS AND LAYING — Dimension stones shall be laid with joints of the widths specified or directed, and shall be lowered carefully into beds of mortar so as not to mar the exposed faces. All dimension stones shall be accurately set, with joints of uniform widths at their faces. All joints shall be completely filled with mortar and finished flush with the faces of the stones as the stones are laid, unless recessed joints are ordered, except the end joint in copings and such other joints as directed, which may not be filled with mortar, but may be left open to allow for expansion and contraction. Steps shall be given a slight camber and pitch, if directed. Dowels, anchors and other metal connections shall be acceptably cemented into holes in the stones where shown on the drawings or directed.

CLASSIFICATION — Dimension stones shall be classified for payment as follows:

Class A shall include stones of specific form and dimension, having a thickness measured back from the neat face less than one-third their least face dimension, and laid with bed joints normal to the face of the dam or other structure.

Class B shall include all stones, not included in Class A, of specific form and dimension which require rectangular blocks having a minimum cross-section of less than 2 sq.ft.

Class C shall include all stones, not included in Class A, of specific form and dimension which require rectangular blocks having a minimum cross-section of 2 sq.ft. or more.

MEASUREMENT AND PAYMENT—The quantities of dimension stones to be paid for shall be the number of cubic yards actually placed in the work, measured as hereinafter prescribed. In measuring all dimension stones for payment, including curbing, the dimensions taken for each stone shall be those of the least rectangular form which will just inclose the neat lines of the stone. Portions of dimension stones which project beyond the prescribed lines into the backing masonry will not be paid for as dimension stones, but as backing. The prices stipulated for dimension stones in Items 29, 30 and 31 shall include all expenses incidental to drilling holes for anchors and dowels, laying and protecting the stones, making and finishing the joints, and the furnishing of all materials, except cement and metal ordered to be embedded in the masonry.

FACE AND JOINT DRESSING OF DIMENSION STONES

WORK INCLUDED—The exposed faces of dimension stones shall be quarry-faced, bull-pointed, rough-pointed, fine-pointed, or patent-hammered, four, six or eight cuts to the inch, or have rounds, moldings, drips or chisel drafts cut in them as directed. Beds and joints shall be dressed to permit designated widths of joints.

GENERAL REQUIREMENTS—In every case, face dressing shall be equal in quality and appearance to that shown on the model or models erected to establish a standard.

QUARRY FACING—Quarry faces shall be bold and have the projections directed beyond the arrises, in general not less than 2 in., and not required to exceed 8 in. They shall be free from lewis holes and tool marks, and the arrises shall be pitched to true, well defined lines.

BULL-POINTING—Bull-pointing may be required for some of the architectural members of the dam. It shall be a very bold, coarse and inexpensive cutting, in scale with the magnitude of the structure. It is not intended that more than the least amount of cutting necessary to bring the stones to a rough approximation of the shapes defined by the ideal lines shown by the drawings shall be

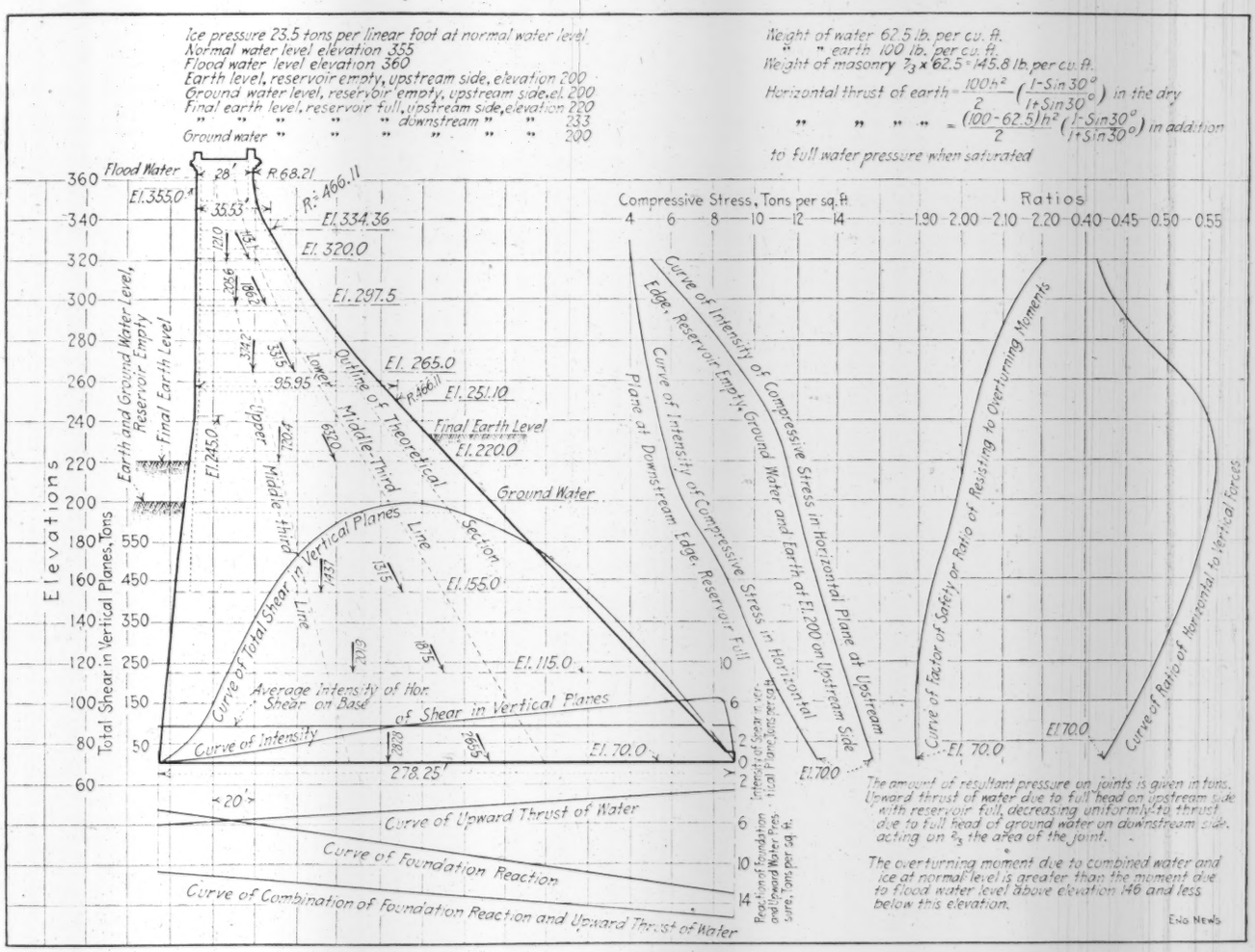

Ice pressure 23.5 tons per linear foot at normal water level
Normal water level elevation 355
Flood water level elevation 360
Earth level, reservoir empty, upstream side, elevation 200
Ground water level, reservoir empty, upstream side, el. 200
Final earth level, reservoir full, upstream side, elevation 220
" " " " " downstream " " 233
Ground water " " " " " " 200
Weight of water 62.5 lb. per cu. ft.
" " earth 100 lb. per cu. ft.
Weight of masonry 7/3 x 62.5 = 145.8 lb. per cu. ft.
Horizontal thrust of earth = 100h²/2 (1-Sin30°/1+Sin30°) in the dry
" " " " = (100-62.5)h²/2 (1-Sin30°/1+Sin30°) in addition to full water pressure when saturated
Flood Water
El. 355.0
El. 334.36
El. 320.0
El. 297.5
El. 265.0
El. 251.10
Final Earth Level
El. 220.0
Ground Water
El. 155.0
El. 115.0
El. 70.0
278.25'
Earth and Ground Water Level, Reservoir Empty
Final Earth Level
Outline of Theoretical Middle Third
Lower Middle Third
Upper Middle Third
Line
Section
Elevations
Total Shear in Vertical Planes, Tons
Curve of Total Shear in Vertical Planes
Average Intensity of Hor. Shear on Base
Curve of Intensity of Shear in Vertical Planes
Curve of Upward Thrust of Water
Curve of Foundation Reaction
Curve of Combination of Foundation Reaction and Upward Thrust of Water
Reaction of foundation and Upward Water Pressure, Tons per sq. ft.
Compressive Stress, Tons per sq. ft.
Ratios
Curve of Intensity of Compressive Stress in Horizontal Plane at Upstream Edge, Reservoir Empty, Ground Water and Earth at El. 200 on Upstream Side
Curve of Intensity of Compressive Stress in Horizontal Plane at Downstream Edge, Reservoir Full
El. 700
Curve of Factor of Safety or Ratio of Resisting to Overturning Moments
Curve of Ratio of Horizontal to Vertical Forces
El. 70.0
The amount of resultant pressure on joints is given in tons. Upward thrust of water due to full head on upstream side with reservoir full, decreasing uniformly to thrust due to full head of ground water on downstream side, acting on 2/3 the area of the joint.
The overturning moment due to combined water and ice at normal level is greater than the moment due to flood water level above elevation 146 and less below this elevation.
Eng News

FIG. 8. MAXIMUM SECTION OF KENSICO DAM SHOWING RESULTS OF INVESTIGATION FOR VARIOUS CONDITIONS

done, so that when viewed at a distance of several hundred feet, the general effect will be a coarse texture with a suggestion of the shapes. Stones shall be full and true to shape at their ends, and such arrises as are shown on the drawings or required shall be properly defined; rusticated also shall be truly outlined on all sides and at their corners; but between these limits the faces of rusticated stones may vary from the

each one shall be known as a member of a molding, except that where curvature of a whole or part of a molding is continuous in one direction through more than 90°, each portion of 90° curvature, and such additional remaining part of less than 90° shall, if not over 3 in. wide, be considered as a member of a molding.

BEDS AND JOINTS—Beds and joints shall be dressed for distances shown on the drawings or directed, not less than

Beds and joints larger than ¼ in.

MEASUREMENT AND PAYMENT FOR FACE AND JOINT DRESSING OF DIMENSION STONES

EXPOSED DRESSED FACES — The quantity of dressing of exposed faces to be paid for, shall be the number of square feet in the superficial area of the finished faces, actually done according to the drawings and directions, within the

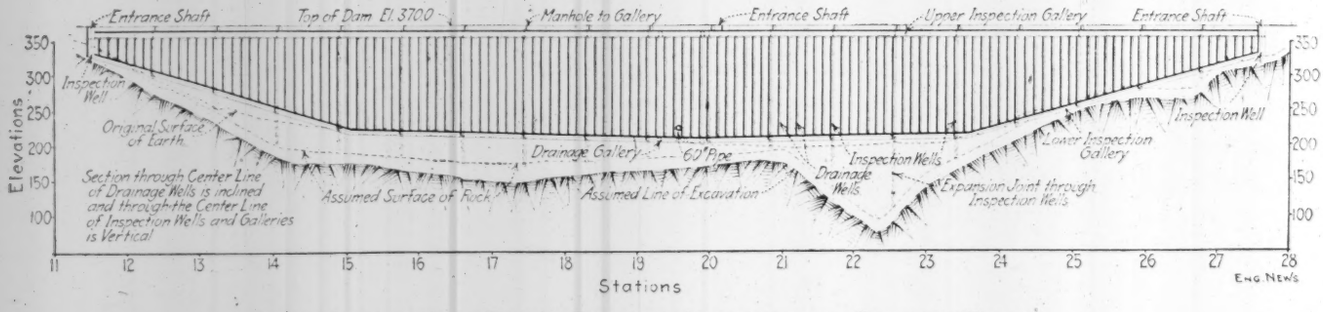


FIG. 9. LONGITUDINAL SECTION THROUGH INSPECTION AND DRAINAGE WELLS

ideal profile by as much as 1 in. back and 4 in. beyond, and for other stones by as much as 3 in. either way. Arrises and angles between adjoining faces of stones, wherever permitted, need not be sharply cut.

FACE DRESSING OTHER THAN QUARRY FACING AND BULL-POINTING—For the other kinds of dressing, faces shall be uniform, and without projections or depressions other than those caused by tooling. The arrises and angles in rusticated and sinkages may generally be roughly rounded, but all others shall be true and well defined, and those of copings, steps, sills, jambs, lintels and similar stones, shall be sharp and smooth. The faces of stones which are to be carved subsequently under other contracts shall project to prescribed distances beyond the arrises.

DRIPS, CHISEL DRAFTS AND ROUNDS—A drip, not more than 1 in. wide, shall be cut in such stones as directed. A chisel draft, not over 3 in. wide, if ordered, shall be cut on quarry-faced stones. Rounds, not over 3 in. in width, and having a curvature of not more than 90°, shall be pointed or patent-hammered, if and as directed, on the exposed edges of dressed stones.

MOLDINGS—Where directed, dimension stones shall have moldings with straight lines and perfect profiles cut in them. For the purposes of this contract, a molding shall mean a continuous face shaped into regular longitudinal grooves or projections, which form in cross-section, curves, straight lines or angles.

4 in. back from the neat face, to surfaces that will permit the designated width of joint. No cavities or hollows more than 6 in. across and ½ in. deep, and nearer than 3 in. to an arris, or whose aggregate area amounts to more than one-quarter of the area of the joint surface, will be permitted on stones dressed for ¼-in. joints; for thicker joints cavities will not be permitted which will interfere with the proper filling of the joint with mortar, or which will jeopardize the strength of the stone or prevent satisfactory setting. Back of the limits of the dressed joints, the stones may taper not more than one-twelfth of the thickness of the finished face for each foot of average depth. The entire end joint surfaces of stones, if directed, for copings, parapets and similar places, shall be dressed for ¼-in. joints. Bed and joint surfaces on some stones shall be dressed for ½-in. joints where directed. All other bed and joint surfaces of dimension stones shall be dressed for joints not exceeding 1 in.

CLASSIFICATION — Face and joint dressing shall be classified for payment as follows, and the prices stipulated shall include all expenses incidental to shaping and dressing the faces and the beds and joints of the stones:

Quarry-faced work.
Bull-pointed work.
Rough-pointed work, including roughing out for carving.
Fine-pointed work.
Patent-hammered work, four cuts to the inch.

prescribed limits, with allowances as hereinafter specified, except that faces which with allowances amount to less than 1 sq.ft. in area shall be measured for payment as having an area of 1 sq.ft. For the purpose of estimating allowances, an exposed face shall be considered as bounded by the joint faces and by its intersections with the surfaces of the least rectangular form which would just inclose the stone.

FACES RECEDING FROM RECTANGULAR FORM—The entire area of all bevels, batters, curved faces, sinkages, moldings and faces from which rusticications and moldings project, which are cut back so as to recede from the plane surface of the least rectangular form which will just inclose the neat lines of the stone, shall be measured for payment as follows: Faces receding at any one point 6 in. or more from each of two adjacent sides of the least rectangular form shall be measured for payment as having an area of twice their actual superficial area. All other faces receding at any point 1 in. or more from the nearest side of the least rectangular form, shall be measured for payment as having an area one and one-half times their actual superficial area. Faces receding less than 1 in. from the nearest side shall be measured for payment as having only their actual superficial area.

STONES HAVING OPPOSITE EXPOSED FACES—Where stones have exposed dressed faces on two opposite sides, each such opposite side for which measurement for payment in addition to

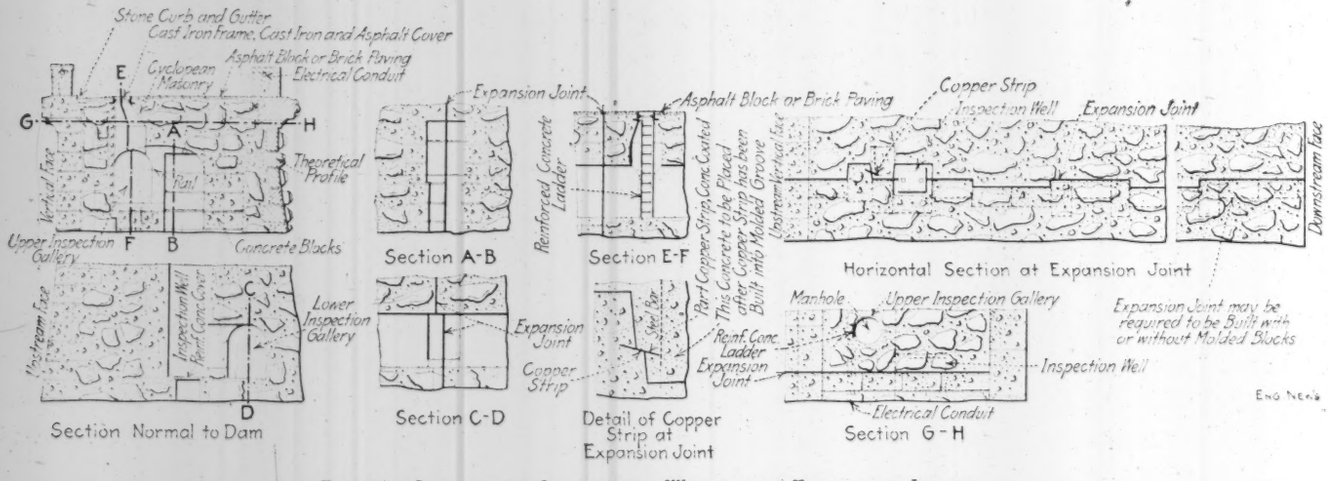


FIG. 10. DETAILS OF INSPECTION WELLS AND EXPANSION JOINTS

Each molding shall be considered as divided into separate longitudinal faces by the straight lines, which in cross-section pass through the points where curves begin, end, or reverse in direction, or where lines intersect. Where two or more such faces are adjoining, and no one of them is over 3 in. wide,

Patent-hammered work, six or eight cuts to the inch.
Rounds not more than 3 in. wide.
Members of moldings not more than 3 in. wide.
Drips not more than 1 in. wide.
Chisel drafts not more than 3 in. wide.
One-quarter inch beds and joints.

that for the actual superficial area has not been otherwise specified, shall be measured for payment as having an area one and one-quarter times the actual superficial area. Where stones are shaped as columns, balusters or similar forms, and have exposed dressed faces on all sides except the ends, or on all

sides and on either or both ends, payment for one-quarter of the actual superficial area of the dressed faces will be made in addition to other specified payments; portions of such stones having perimeters of cross-section, normal to the axis, of less than 32 in., shall have their faces measured for payment as though the perimeters of cross-section were 32 in., and an additional payment for one-quarter of the area so measured will be made.

DRESSED FACES LESS THAN 8 IN. WIDE—Each dressed face which is less than 8 in. in its least dimension and which is not a continuation of a dressed joint, and for which measurement for payment in addition to that for actual superficial area has not otherwise been specified, shall be measured for payment as having a least dimension of 8 in.

QUARRY-FACING—Quarry-facing shall be measured for payment as the area included within the arrises, disregarding chisel drafts and the projections beyond the neat lines.

ROUNDS, DRIPS, CHISEL DRAFTS AND MEMBERS OF MOLDINGS—Rounds, drips, chisel drafts and members of moldings shall be measured for payment as the actual number of linear feet cut as directed, this payment being in addition to other specified payments under the proper item of face dressing.

ROUGHLY SQUARED STONE MASONRY

WORK INCLUDED—Roughly squared stone masonry may be used for facing portions of the dam, for facing spandrel walls and abutments of bridges and at other places.

GENERAL REQUIREMENTS—Stones, unless otherwise permitted, shall be roughly squared so as to have nearly rectangular faces with approximately straight edges. The exposed faces of stones shall be acceptably bold quarry faces, free from lewis holes and tool marks, with the projections directed; no portion of any stone shall project less than 1 in., measured normally, beyond the face of the mortar joints, and some point of each stone shall project at least 3 in. Projections will not be required to exceed 12 in. At least 20% of the face area shall be headers extending at least 3 ft. back from the neat face; but if any dimension stones acting as headers be set in any area of roughly squared stone masonry, they shall be included when reckoning the per cent. of headers. No stones shall have exposed faces less than 2 sq.ft. in area and at least 50% of the exposed face of such masonry in the dam shall consist of stones each having a face area greater than 6 sq.ft. In general, as many stones as possible shall be as large as it is practicable to handle. Stones shall extend into the dam or other structure to acceptable depths, but in general will not be required to extend more than 2 ft. measured normally from the face of the mortar joints.

BOND—The arrangement of headers and stretchers shall be as directed. The face bond between any two stones shall not be less than one-third the horizontal length of the face of the smaller stone, but vertical joints between stones, where permitted, may be continuous for not more than 6 ft.

LAYING—All stones shall be set so that the face of the mortar in the joints will be about 1 in. back from their average edges, or the mortar shall be raked out to a depth of 1 in. and neatly finished as the stones are set. The tops of these projecting portions of the stones shall, as far as practicable, be horizontal or slope slightly downward so as to shed water. Each stone, unless otherwise directed, shall be laid upon its longest side in a prepared bed of mortar or mortar and spalls, in such manner as to assure the filling of the bed joint with mortar. The end joints shall then be completely filled with mortar. No spalls shall show upon the face.

JOINTS—Joints shall average not more than 2 in. and nowhere shall be greater than 3 in. in width at the face, but they will not be required to be less than 1 in. at any place. All joints shall be finished in a workmanlike manner as the stones are laid.

MEASUREMENT AND PAYMENT—The quantity of roughly squared stone masonry to be paid for shall be the number of cubic yards actually placed in the work in accordance with the drawings and directions, measured as having the area, including joints but excluding dimension stone headers, if any, within the limits ordered, and a depth of 2 ft. from the face of the mortar joints, normal to the theoretical profile. Portions of stones projecting beyond the payment line into the backing masonry will be paid for as part of the backing. The price stipulated shall include all labor and expenses incidental to shaping, facing and laying masonry, and furnishing all materials except cement.

Kensico dam is being built by the Board of Water Supply of the City of New York, J. Waldo Smith, chief engineer; Merritt H. Smith, deputy chief engineer. Messrs. John R. Freeman, Wm. H. Burr and Frederic P. Stearns advised as consulting engineers. Field work is in charge of Frank E. Winsor, department engineer, Southern Aqueduct Department, and Wilson Fitch Smith, division engineer. The contract and designs were prepared under the writer's immediate supervision by Charles E. Gregory, designing engineer, and H. Lincoln Rogers, architect, and Percy C. Barney, principal assistant engineer.

Hypochlorite Sterilization and and Typhoid at Kansas City, Mo.*

By Walter M. Cross†

Owing to the fact that there have always been present in the sedimented but otherwise untreated Missouri River water, germs whose positively known origin and normal habitat is the intestinal tract of man and animals, the question has been an open one in the minds of many sanitary engineers as to whether or not the city's water supply should not be unreservedly condemned for drinking purposes unless it has first been boiled or sterilized. Indeed there have been occasions when the city chemist has urged upon all of the citizens the advisability of boiling all river water before drinking it.

Then to make matters much worse, many people began to use the clear and sparkling waters that issue in many places from springs and that can be pumped from wells. These wells and springs throughout the settled districts of the city, although of very attractive appearance, have been almost invariably the cause of disease at some time, and in most instances are at least ten times as badly contaminated as was the river water. Thus it happened that the city's water, on account of its questionable character, became a possible direct and certainly an indirect cause of disease.

During 1908-9 some experiments were made with ozone to determine, if possible, some practical method of sterilizing our entire water supply, at least as far as the destruction of germs likely to become the cause of disease, is concerned. These experiments indicated that although it was possible to sterilize to a satisfactory degree all of the city water, the expense of installing and maintaining the apparatus would be prohibitive.

During the early part of the fall of 1909 the Fire and Water Board of Kansas City ordered the writer to make a tour of inspection of several cities known to have the hypochlorite sterilization system in satisfactory operation. It was found that the effect was fully as good as reported, and that the method of installation of the system was both easy and cheap. It was seen that calcium hypochlorite containing 37% of available chlorine, and costing a little over 15½c., was sufficient to treat a million gallons of a moderately polluted water. The Fire and Water Board then immediately took steps to install this system of treatment to improve the entire municipal water supply of Kansas City.

In view of the fact that a great many cases of typhoid fever are imported and that there are still open and being constantly used numerous springs, wells and cisterns known to be contaminated with sewage, the result has been all that could have been anticipated. During the year 1910 no specific attempt was made to destroy pathogenic germs in the municipal water supply. During the year 1911 the hypochlorite process was used throughout the year. Table I shows the number of deaths from typhoid fever month by month during those years:

TABLE I—DEATHS FROM TYPHOID FEVER AT KANSAS CITY, MO.

Month	1910	1911
January	17	6
February	21	6
March	10	4
April	8	3
May	3	4
June	7	1
July	9	5
August	8	6
September	6	3
October	6	8
November	4	6
December	8	9
Totals	107	61

With this great source of danger from infection by the municipal water supply removed, it is possible for the Health Commissioner to so enforce the abandonment of questionable sources of water supply and regulate sanitary conditions as to make the occurrence of typhoid fever and most intestinal diseases an uncommon thing in Kansas City.

Table II gives some germ counts on the city water supply, showing the improvement of the water by the hypochlorite process:

TABLE II—GERMS IN 1 C.C. OF WATER AT KANSAS CITY

	Before treatment		After treatment	
March	Quindaro River	Clear water basin	Hydrant at city hall	B. Coli
20	10,000	1,200	75	0
21	8,000	1,800	70	0
23	4,000	800	100	0
24	10,000	500	55	0
25	8,000	600	90	0
27	8,000	400	25	0
28	5,000	260	20	0

Overheating Near the South Pole was the unexpected cause for the abandonment by Capt. Scott, the English explorer, of his motor-driven sledges. The system of propulsion (described in the technical press early in 1910) proved satisfactory, but the air-cooled gasoline engines became overheated even at latitude 80½°.

*Abstract of a paper presented at the annual meeting of the Illinois Water Supply Association, at the University of Illinois, Mar. 5, 6, 1912.

†City Chemist, Kansas City, Mo.

The Directograph Pitometer

A device which has been introduced recently for measuring and recording the flow of liquids in pipes, and which has the special feature of showing and recording its results on a direct-reading chart, is known as the directograph pitometer. It is used for recording the rate of flow as measured by pitot tubes, venturi meters, or other devices in which the desired result is obtained by the measurement of the difference between two pressures. As described in this article and shown in the accompanying cuts it is designed for use in connection with a pitot tube for measuring the rate of flow in water mains.

The instrument and its connections to the main are shown in Fig. 1. The pitot tube is of a standard form, having two orifices or pressure openings and connected with pressure tubes passing through a metal casing of such form and length as will permit its insertion into a water main through a 1-in. opening and will allow it to be set at any point across the diameter of the pipe. These two pressure tubes are connected by means of rubber tubes or pipes to a glass U-tube containing a liquid slightly heavier than water. Thus they form a good serviceable indicating flow meter for use in cases where a short test measurement is desired on any pipe line. When it is desired to obtain a continuous record of the rate of flow, from which the total output of any line (as for example a pumping-station discharge) can be computed, the recording device is connected in parallel arrangement with the U-tube, as shown in Fig. 1. This method of connection allows the instrument to be used either for indicating or recording the rate of flow, and furnishes means for checking the adjustment of the recorder.

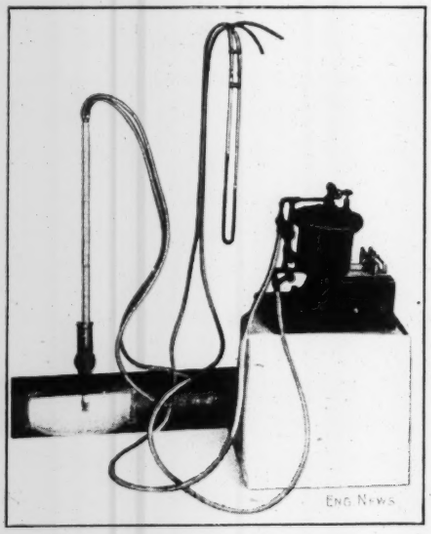


FIG. 1. THE DIRECTOGRAPH PITOMETER WITH PITOT TUBE AND CONNECTIONS TO RECORDING DEVICE

(W. R. Brown, Inventor, Water Works Specialty Co., Chicago; Maker.)

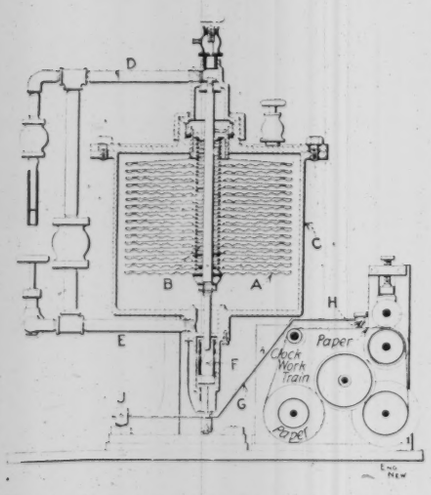


FIG. 2. SECTIONAL ELEVATION OF THE RECORDING APPARATUS OF THE DIRECTOGRAPH PITOMETER

The recorder is shown by the sectional drawing Fig. 2. It consists of a collapsible metal box or bellows *A*, built up of flat and corrugated diaphragms. This contains a helical spring *B*, made of phosphor-bronze wire, and is inclosed in a cylindrical brass case *C*, in such a manner as to form two separate pressure chambers. The upper chamber or interior of the collapsible box *A* is connected to the upstream or impact orifice of the pitot tube by the tube *D*, and the lower or outer chamber is connected to the downstream or trailing orifice by the tube *E*.

If there is no flow in the main, the collapsible box will stand at a normal position of rest, as the pressure in both chambers will be equal. But if the water in the main is in motion, the pressure in the upper chamber is greater than in the lower chamber and the bellows is extended or expanded a distance which is dependent upon the amount of difference between the pressure in the two chambers and the resistance of the helical spring. This thrust is transformed by means of rollers and cam *F* into a rotary motion, and is transmitted through a frictionless packing gland to a pen arm *G*. This arm carries a pen *H*, which marks on a continuous paper chart, and this is counterbalanced by the weight *J*.

The chart (Fig. 3) is made in the form of a continuous sheet or ribbon and has parallel lines indicating feet and fractional feet of velocity per second. A sample of a chart with record is shown in Fig. 3, and it will be noted that the transverse lines or hour-divisions are curved to agree with the radial movement of the marking pen. The mechanism is so constructed that the distance traveled by the pen is directly proportional to the increase in velocity, thus permitting the use of a polar planimeter to obtain the average rate of flow for any period of time. This form of chart is especially desirable, since average rates and therefore total amounts for given periods can be obtained from it with a minimum of office work. In fact, where the flow is fairly steady, as at many pumping-station outlets, the average rate can be obtained by inspection without the use of the planimeter.

The portability and convenience of installation combined with the direct-reading chart are claimed to make this meter specially adapted for the use of the water-works engineer, as by its use he can determine (1) the distribution of supply along the lines of the various feeder mains in the system, (2) the velocities under which these feeder mains are working, (3) the amount of water used in any particular section of the distribution system, and (4) the actual output of the pumps and consequently the efficiency of same.

In making field surveys, the ease with which the instrument is installed and the small expense involved enable the operator to proceed much more rapidly and to carry on the investigation to include much smaller limits of territory than he would be able to do by the use of meters which require cutting into the main and the resultant permanent installation of the meters or expensive bypass connections. Further, the number of meters required

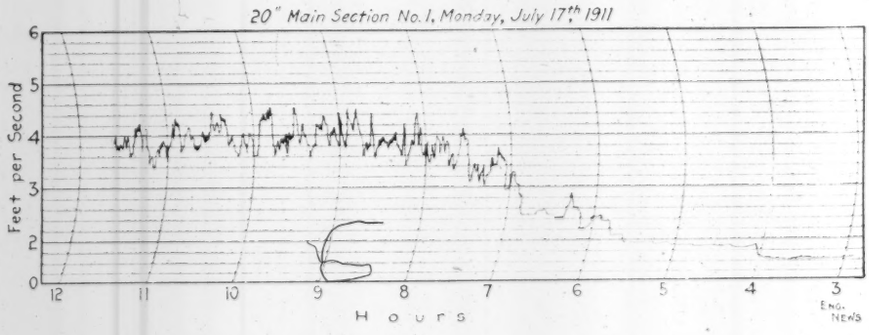


FIG. 3. SECTION OF CHART MADE BY THE DIRECTOGRAPH PITOMETER

(This shows a record on a 20-in. main at Milwaukee, Wis., from 3 a.m., till noon, July 17, 1911.)

to carry on a survey is limited to the number of points at which simultaneous readings are desired, regardless of the size of main, since the same instrument can be used on any main, from a 4-in. to the largest size in the system.

When used as a master meter to determine the output from a pump or other source of supply, it offers the same advantages as to convenience and cheapness of installation. When used on a pump it can be equipped with a special scale attached to the U-tube so calibrated as to read in terms of revolutions per minute, thus making a convenient indicating instrument with which to check slip of the pump by a very simple comparison of readings on the scale with readings of the revolution counter. The care of this instrument is much the same as that required by any recording gage and therefore it can be operated at a station by the engineer or his assistant who has charge of the other various gages about the plant without additional expense of expert trained service.

This device was designed by W. R. Brown, of Chicago, and was tried by him with satisfactory results on both pumping-station and field-survey tests at Milwaukee, Wis., during the past summer. Fig. 3 is a reproduction of one of the charts obtained during these tests. Patents on this invention were obtained by the Water Works Specialty Co., 35 So. Dearborn St., Chicago, and it is now being manufactured by that company. For information we are indebted to both this company and to Mr. Brown.

Water-service Connections from the street main to the property line of consumers must be made at the expense of water companies in Idaho, according to a decision handed down by the U. S. Supreme Court, on Apr. 1, 1912. The Consumers' Water Co., of Cœur d'Alene, Idaho, refused to make a service connection for Albert L. Hatch unless he would comply with the regulations of the company by either paying $8.50 for the connection or comply with some other regulation designed to recoup the company for the expense. Hatch brought an action for mandamus in the Idaho Supreme Court, which ordered the company to make the connection without charge and supply water through it at its regular monthly rate. The company appealed to the U. S. Supreme Court, with the result just noted. From the decision of the higher court it appears that the statutes of Idaho, as enacted in 1887 and revised in 1910, virtually cover just such a case as this. Under the earlier statute the Idaho Supreme Court, as cited in the opinion before us, decided in favor of a consumer in an apparently similar contention in a suit which involved the Pocatello Water Co. In the Cœur d'Alene case the U. S. Supreme Court held that the fact that Hatch lived on an ungraded street, and that the company could not be required to lay mains on such streets, had no bearing in this case, because the main was already laid when Hatch applied for the connection.

Reducing the Boiler Hazard

By Alex. M. Gow*

One of the largest boiler-inspection and insurance companies in the country has recently published a statement† covering a period of 41 years, showing the number of boilers insured, inspections made, defects discovered, etc. A careful study of this report warrants the conclusion that the hazard incident to the operation of steam boilers has very materially decreased in the last 20 years.

For purposes of comparison, it is needless to go back further than 1890, or a period of 21 years, because previous to that date there were undoubtedly many boilers in service that would be condemned today and in addition we would get into an era of lighter pressures. From the figures given in the statement certain curves have been plotted, shown in Figs. 1 and 2. The curves in Fig. 1 show the

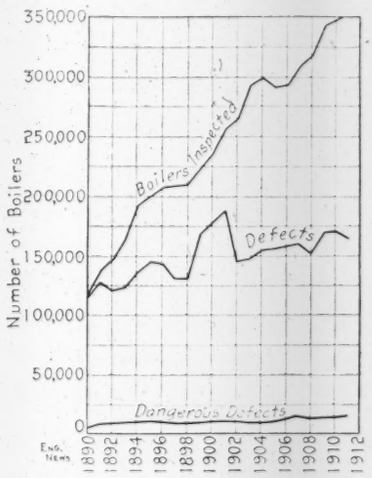


Fig. 1. Curves Showing Relative Increase in Boilers Inspected and Defects Discovered: 1890-1911

number of boilers inspected, the defects and the dangerous defects reported. Fig. 2 shows the defects and dangerous defects for each one thousand boilers inspected.

While the number of boilers inspected has steadily increased from 118,098 in 1900 to 352,674 in 1911, the number of defects reported has increased only from 115,821 to 164,113 in the same period. Inspections are undoubtedly more rigid than they were twenty years ago. Nevertheless, in proportion to the number of boilers inspected, defects have been reduced by more than one-half.

More striking still is the reduction in dangerous defects. While the number of boilers inspected has increased nearly in the ratio of 1 to 3, the dangerous defects have increased from 9387 to 17,410, or a ratio of but 1 to 1.85. In 1890, for each 1000 boilers inspected, there were 981 defects and 79 dangerous defects reported. In 1911, for each 1000 boilers inspected, there were 467 defects and only 49 dangerous defects reported. Putting it another way, in 1910, 98% of the boilers showed defects and 8% dangerous defects. In 1911 only 47% showed defects and only 5% dangerous defects.

Of course, the classification of defects and dangerous defects is a matter for the judgment of the inspectors; but the actual improvement in conditions is certainly better than the figures show, rather than worse. It is noticeable, however, that in the reduction of dangerous defects there has been no improvement for 12 years. The curve of dangerous defects in Fig. 2 is nearly straight for that time. But it is probable that increased rigidity of inspection would now class as "dangerous" a defect that would not have been so reported years ago.

The hump in the curves of defects, occurring at about 1900, is curious and

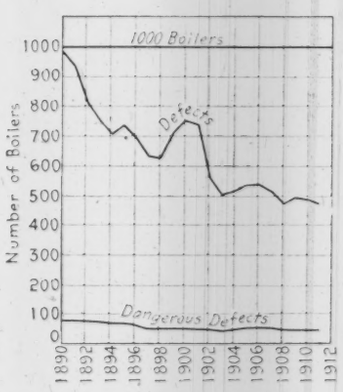


Fig. 2. Curves Showing Defects and Dangerous Defects per 1000 Boilers: 1890-1911

might be taken to indicate an unusual activity on the part of inspectors; and the further fact that the curve drops to its normal position about 1903 may mean that the inspectors dropped back into their old ways or that a decided improvement was effected in the condition of the boilers. A more probable explanation, however, is that the business activity about 1900 put into service every available boiler and then crowded them to their capacity. With the business reaction of 1903 boiler plants were fixed up and the cripples put out of commission. There is little question that industrial hazard bears some sort of a direct relation to output. When business is booming look out for accidents.

There is so much being said and done at present upon the whole subject of safety to employees that it is interesting to know from data like this that the boiler hazard is on the decline. There is room, however, for still further improvement.

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*Assistant Chief Engineer, Oliver Mining Co., Wolvin Bldg., Duluth, Minn.

†The Hartford Steam Boiler Inspection & Insurance Co., in the "Locomotive" for January, 1912.

FIELD AND OFFICE

Thawing Out a Frozen 8-in. Pipe Sewer at Beloit, Wis.

By R. R. Caldwell*

About Feb. 20, 1912, I was notified by a householder that there was something wrong with the sewer in his street. A plumber who had been called in to investigate an apparent stoppage in his house sewer had reported an obstruction in the street sewer. On going to a manhole 300 ft. below the house connection I found the intake end of the invert frozen almost full and the outlet end of this invert about half full and frozen. I took my sewer rods and found that I could get a stream through to the next down manhole, 330 ft. distant. The next thing was to get a stream the other way, which I found somewhat more difficult.

Although my experience on such work covers a period of about twenty years and includes about all kinds of sewer troubles, this was my first experience with a sanitary sewer frozen solid. Like other engineers in times of trouble, I began to do some hard thinking. Happening to see a man sawing wood with a steam engine it occurred to me to try steaming out the ice. I asked the man if he would try the job for me. He said he would be very glad to do so.

I got about 100 ft. of ⅜-in. iron pipe, cut into 4-ft. lengths and some rubber hose. The pipe was taken down into the first manhole, connected to the boiler by means of the hose and the steam turned on. The boiler was of 12 hp. and we started with 80 lb. of steam. I applied the steam to the up end in the manhole and thawed out the sewer with the first 4-ft. length of ⅜-in. pipe and put on another 4-ft. length and so on until I got about 20 to 25 ft. Then the steam condensed and the last 4 ft. went slowly and at 25 ft. it was a failure.

Then I had to resort to thinking again. By this time, however, voluntary advice was plentiful, for a steam engine in the street blowing off steam had attracted a crowd, every member of which knew just how to solve my problem. Having learned by experience to rely upon my own wits in such cases, my further thinking hit upon this plan, which brought success.

We got 100 ft. of ¾-in. steam hose and reduced the opening to ¼ in. at the end. We also got 100 ft. of ⅛x2-in. tire iron and wired the hose to the end of it. We started the steam and at the same time started the tire iron and hose up the sewer. We ran the combination forward at the rate of 1½ ft. per minute for about 60 ft., then it began to slow up. The next 10 ft. took 40 minutes; the next 10 ft. 50 minutes, and at 80 ft. it stopped.

Thinking was again in order. I finally concluded that it would be best to cut a hole in the pavement 160 ft. up from our start and did so. Our frost was down to from 5½ to 6½ ft., and as is generally the case, trouble will start with a sewer in about the worst place possible, which in this case was a newly paved tar-macadam street, where the sewer was about 5 ft. deep.

Cutting a hole to the sewer took a long time. (Two men could bury one man in this under such conditions in about 48 hours.) We eventually reached the sewer, cut a hole therein, moved our engine and started down hill with our steam, tire iron and the hose.

This time we had the best of success. The steam heated the water as the ice melted and we shoved the hose at the rate of 2 ft. per minute on the entire 80 ft. We then turned the steam up hill and thawed 80 ft., which again seemed to be as far as the steam would work. This took 55 minutes and carried us to another manhole. All told, we thawed out about 780 ft. in this manner.

Everything counted in, the job cost on an average of about 8c. per ft. The man and steam boiler cost us $5 per day for four days of eight hours per day, and we used about one-fourth of a ton of coal a day, at $4 a ton, which was not included in the $5 per day. This was the least of the expense. Cutting three holes in the pavement took two men for four days, at $1.75 per day, making a total of $14. The material bought for the work cost a total of $20 and included hose, iron pipe and wood to thaw out the pavement. Altogether, the total cost was $64, not including portions of four days of my own time. We did not keep the boiler going continuously.

The temperature for 20 days before the stoppage by ice was reported ranged from 0° to −20° F.

The sewer was 8 in. in diameter, laid on a 0.4% gradient. It was a sanitary sewer with which two catch basins *happened* to be connected and was frozen *solid* for 780 ft. The flushtank at the extreme upper end was frozen full, as well as the water supply to the tank.

Finally, I cannot see that the sewer pipe itself was in any way affected by having been frozen solid, but I had no means of judging except where the pipe was exposed by digging down to it, and what I could see at the manholes.

*City Engineer, Beloit, Wis.

Steel-Reinforced Wood Transmission Line Poles

By L. W. R. Allison*

The Pacific Light & Power Corporation, of Los Angeles, has recently rebuilt a 15,000-volt 10-mile transmission line from its power plant at Redondo Beach to a substation of the Pacific Electric Ry. Co., at Dominguez Junction, to provide for the increased power requirements of interurban travel. The main line consists of two parallel lines of wood poles, two double crossarms to the pole, each carrying three 312,000-circ.mil. cables. The terminal poles had to be of special design (1) to carry the exceptionally heavy lines, (2) to permit of ready entrance to the station at right angles to the line, and (3) to prevent interference of the wires.

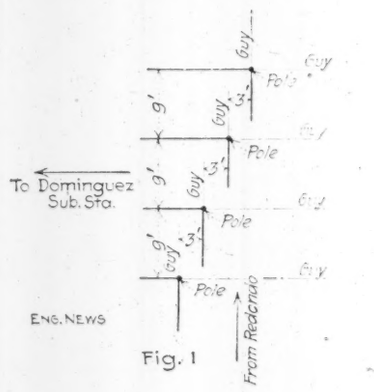

Fig. 1

Four wood poles were used, set staggered as shown in the accompanying diagram to permit guying in both directions. They were of selected stock 55 ft. long, with 15-in. butts and 9-in. tops. Each was reinforced for its entire length with two 6-in. 8-lb. steel channels, flanges inward. The channels were single 55-ft. lengths and were fastened to the poles in the field by ¾-in. bolts on centers varying from 24 in. at the butt to 12 in. at the top. Each pole had a single crossarm—a 6-in. 8-lb. channel 6 ft. long, at right angles to the pole reinforcement, with the web against the pole, and the center 6 in. below the pole top. It was held in place by a 3x½-in. wrought-iron strap bolted to the channel through the pole at the center and directly to the channel at each side 11 in. from the ends. Also two regular 26x1¼-in. wrought-iron braces were used in the usual manner. This is shown in Figs. 2 and 3.

The twelve leads from the station terminate, three to a pole, on single

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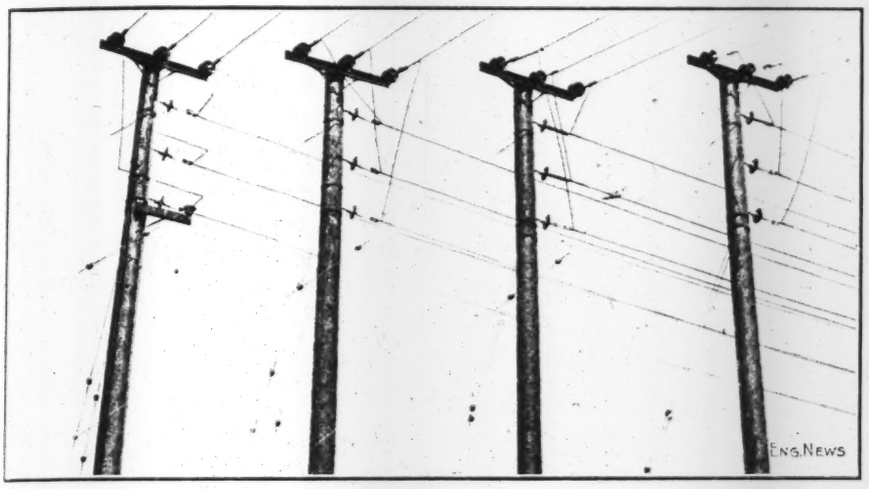


FIG. 2. TERMINAL POLES OF A 15,000-VOLT RAILWAY SERVICE LINE; REDONDO BEACH, CALIF.

porcelain suspension-type insulators spaced 3 ft. apart vertically, attached to the web of the channel by 5/8-in. eye-bolts passing through both channels and the

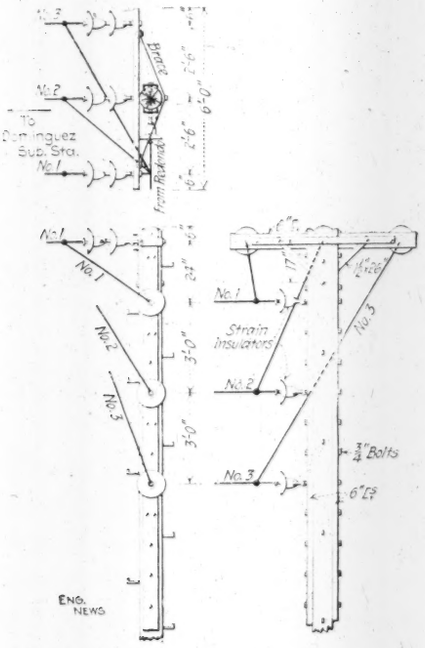


FIG. 3. STEEL-WOOD CONSTRUCTION; TRANSMISSION LINE TERMINAL; PACIFIC LIGHT AND POWER CORPORATION

pole, and having the eye welded together after the eye of the insulator bolt is inserted. The transmission lines leave the crossarms on similar insulators, but with two disks in place of one.

Lucky Contractors—That the business of construction contractors is something of a gamble appears to be substantiated if a report from Scranton, Penn., is true. The building there of a trunk sewer in tunnel, 70 ft. below the surface, is said to have opened a 6-ft. vein of anthracite. The contractors are reported "jubilant."

Methods of Patching and Repairing Plain and Reinforced Concrete*

(1) WET METHOD—The surface of the old concrete is thoroughly roughened, cleaned and drenched with water and covered with a cement grout. Then the new concrete is mixed to a sloppy consistency and applied, being held in place till set by forms, as required.

From the reports received it appears that many successes and many failures have followed this method. Where it has been successful, that result probably has been due to painstaking care and expense being employed in the preliminary cleaning and thorough wetting of the old concrete surfaces before the new concrete is applied.

(2) MODERATELY DRY METHOD—The surface of the old concrete is thoroughly roughened, cleaned and drenched with water and powdered with cement or painted with cement grout—after which the new concrete is applied and thoroughly tamped against the old surface. Generally, such patches are kept moist by sprinkling them with water for several days. As to results, the same comment applies here as to the wet method.

(3) DOWEL METHOD—This is a modification of the two preceding methods and is generally used for other than horizontal surfaces, although it may be applied to the latter. The new concrete may have any consistency to meet the particular conditions or the ideas of the engineer in charge, but usually a fairly wet concrete is used. This method includes drilling holes and setting dowels into the old concrete, with projecting ends to engage into the new concrete. Sometimes steel bars or metal fabric are connected to these dowels to further insure permanency of the patch.

This method can be made uniformly successful, but cannot always be applied. Where any considerable mass of new concrete is to be connected to old concrete this is the only safe method to pursue.

(4) WEDGE METHOD—This method includes the cutting out of defective concrete in such a manner as to undercut the surrounding good concrete so that the new concrete will be held in place, when set, by the shape of the binding edges of the old concrete.

The difficulty of undercutting concrete so as to secure such edges is too great to make this method applicable in ordinary cases, and, except where a considerable depth of new concrete is to be put in, such undercutting could not be successfully done, as concrete cannot be cut so as to leave sharp, clean edges.

(5) CLEANING WITH STEAM—This is a method of cleaning surfaces of old concrete preparatory to putting on a patch of new, or of cleaning the top of a rough concrete floor before applying the "finish coat." The surface of the old concrete is brushed as clean as possible with stiff brushes and then gone over at least twice with a steam jet—an ordinary piece of ¼-in. gas pipe makes a very satisfactory jet nozzle. This short length of pipe is connected to a steam hose so that it can be readily moved about.

The steam is supplied by the boilers of the mixer or hoisting engines. The steam will clean and heat the concrete surface, leaving it perfectly dry, so it is important to thoroughly drench the surface with water after cleaning with steam. In cold weather it is found advantageous to use hot water. Immediately after the surface is thoroughly cleaned and drenched, the new concrete is applied.

Excellent results have followed the use of this method of cleaning, but it is essential that the surface of the old concrete be thoroughly drenched with water after using the steam jet; otherwise the new concrete will not adhere to the old.

(6) CLEANING WITH ACID—This method includes washing of the surface of the old concrete with a solution of hydrochloric (muriatic) acid (one part acid to two parts water), after which the surface must be carefully and thoroughly washed to remove any free acid.

This is a very commonly applied method of cleaning old concrete surfaces preparatory to placing new concrete against them, and by many is considered the only safe one to use, especially where a "finish coat" is to be put on after the concrete base is set.

CONCLUSIONS

(1) In all cases the surface to be repaired must first be thoroughly cleaned of all loose material, laitance and dust, and the clean, rough, sound concrete ex-

*Abstract of the report of the Committee on Masonry, presented at the annual meeting of the American Railway Engineering Association, at Chicago, Mar. 19-21.

posed to receive the patch. Probably the best method of cleaning is by means of a steam jet.

(2) After cleaning, the surface to be repaired must be thoroughly saturated with water, not simply moistened, but so thoroughly drenched that the old concrete will not absorb water from the new mortar or concrete used in patching. If possible, the surface should be kept covered with water for several hours.

(3) If the repair or patch is to be made on a vertical or sloping surface and is not to be more than 1½ in. thick, the surface of the old concrete, while it is still wet, should be spattered or splashed with a cement grout, following this immediately with a fairly stiff plaster coat of mortar made of the same proportions of cement and sand as was used in the original concrete, but never richer than 1 cement to 2½ sand. This plaster coat should not be thicker than ½ in. and each coat should be forced into the surface, but not dragged with a trowel.

The surface of each coat, except the final coat, should be "scratched" to give a bond for the next coat. This plastering should preferably begin at the top and progress downward, and only enough time be allowed to permit each coat to receive its initial set before the next coat is applied. The final coat should be finished with a wooden float and only enough water used to properly finish the surface. This patch should be kept damp and protected from sun or frost till fully set up.

(4) If the repair or patch or "finish coat" is to be made on a horizontal or nearly horizontal surface, the surface of the old concrete should be slushed and broomed with a thin cement grout, following this immediately with a *wet* mortar made of 1 part cement and 2½ parts sand or granite screenings and of the full thickness required (not less than ½ in. thick, however). When this mortar begins to take its initial set, it should be floated or troweled to such a finish as may be desired.

(5) If the repair or patch is to be made on a vertical or sloping face and is to be more than 1½ in. thick, it will be advisable to embed dowels into the old concrete, as deeply as the thickness of the proposed patch, and spaced sufficiently close together to firmly anchor the patch to the old concrete. The dowels must be wedged into the old concrete and it will be advisable to fasten wires, metal fabric or bars to the dowels, in the case of extensive patching, as an additional safeguard. The patching may then be done with mortar without forms, or with wet concrete supported by forms, depending upon the thickness and the extent of the patch.

(6) If the repair or patch is to be made on a horizontal or nearly horizontal face of considerable thickness, dowels may be used, or the concrete may simply be reinforced by fabric or bars without using dowels—treating the patch as a block of masonry.

(7) Care must be taken not to have thin edges on patches. To avoid this, it may be necessary to cut out sound concrete around a place to be patched, so as to give deep edges to the patch. If possible, the edges should be undercut.

Examples of Concrete Repairing*

Cincinnati, Hamilton & Dayton Ry.—Small imperfections in finish we repair with stiff cement grout immediately after the removal of the forms. We aim to simply fill in small cavities, not by plastering, but by working the grout into each individual cavity, finishing afterward, while the grout is green, by rubbing with a small block of wood.

Larger cavities we fill whenever weather conditions are favorable. In filling such cavities, we aim first of all to thoroughly clean the cavity of all loose stone, cement and dirt. We are very particular about cleaning the cavity, using for the purpose a weak solution of acid, if necessary, later washing all remaining acid from the cavity. We then use 1:2 grout or a lean concrete mixture, mixed rather stiff.

This mixture we are very particular to thoroughly mix, that is, if mixing by hand, where three turnings would suffice for ordinary concrete, we give the material intended for patching some eight or a dozen turnings. In depositing this material in the cavity we seldom use anything resembling a form to retain it in place, but put the material in with trowels in small layers, thoroughly working it into place. If the mixing and depositing is thoroughly done, such repairs will be permanent.

Atchison, Topeka & Santa Fe Ry.—In one small abutment the bridge foreman allowed the weight to be placed upon it too soon, when a wedged shaped piece of the abutment broke out with about half of the bridge seat as the base, and extending down 3 or 4 ft. on the face of the abutment, to the point of the wedge. The broken face was thoroughly roughened up, a form built around it and concrete of the same mixture was very thoroughly tamped into position.

On some reinforced work, the section foreman knocked off a part of the form in connection with his track work, and in doing so knocked off several corners of the concrete. We built up forms around these and thoroughly tamped in a rich mixture of equal amounts of sand and cement, and today the break cannot be detected. Several concrete steps were replaced several years ago by roughening the surface and putting in nails for dowel pins. This was thoroughly tamped with a rich mixture and has never shown any signs of failure.

The secret in patching concrete seems to have been in having a rough surface to work on, and in thoroughly tamping the concrete and putting in dowels or anchors, of any kind, wherever possible; also, in leaving it in the forms until it was thoroughly set.

Lehigh Valley R.R.—A double-track ribbed abutment, supported by piling, broke in two about the center line, and opened up a crack about 1½ in. wide (due to settlement and leaning of piling in foundation). After movement of piling ceased, a V-shaped opening about 12 in. wide and 6 in. deep was cut in the face of the abutment and that opening filled with comparatively dry concrete; after setting of this concrete, the crack was filled with concrete grout. A slight crack due to contraction of concrete appeared along the edges of the V-shaped patch, but is of no consequence. The patch has been in place about 1½ years and is giving satisfaction.

Delaware, Lackawanna & Western R.R.—Where honeycomb or porous places occur in face, due to improper spading or a leak in forms, which allows cement and sand to escape, we require voids to be filled immediately upon removal of forms with cement mortar of same proportions as used in the body of concrete and smoothed with float.

One of the worst defects in concrete faces, and one which rarely develops until some time after the completion of the structure and, therefore, is hard to repair, is caused by the failure to remove the sludge which invariably rises to the surface of wet concrete and forms a slimy coating of variable thickness at the end of each day's work. If this is allowed to remain, and the next day's work deposited thereon, it will later dissolve, leaving a more or less ragged line of demarkation on the surface. While this is inexcusable, it sometimes occurs. We then chisel it out in the form of a dovetail, filling with cement mortar and stone screenings, taking care that mixture is not made too rich; otherwise, there will always remain too great a contrast in color. This should be floated off with wooden float or wire brush so as not to leave a glossy finish. We do not depend on new concrete adhering to old, but cut out the imperfection to sufficient depth and dovetail in the patch.

The only extensive job which we have had to contend with was a concrete bridge consisting of several 33-ft. arches. The concrete in this structure showed signs of either poor cement or acid-bearing water used in making the concrete, with the result that it scaled off in large quantities every year. It was an easy matter to dig down into the solid concrete, but if the solid concrete was exposed for any length of time it again would start to deteriorate. The entire structure was re-

*Some of the engineers furnishing information to the committee stated that they had not obtained satisfactory results with any method of patching. We give abstracts of only a few of the letters.

paired by covering the same completely with 12 to 18 in. of reinforced-concrete, the new concrete being bonded to the old by a large number of dowel pins, which were drilled into the old concrete and fastened in a substantial manner. This repair job has proved to be very satisfactory. To make the work permanent the entire outside of the old structure was excavated and properly waterproofed.

NEW YORK CENTRAL R.R.—In some instances, the surface of the concrete to be repaired has been first roughed up and then kept thoroughly saturated with water for several hours. Then a coating of cement mixed 2:1 is thoroughly troweled on and kept moist for a considerable period of time. A very good way to keep the surface moist is to place several layers of burlap on top of the fresh surface and keep the burlap soaked with water. Where the surface to be repaired is of a considerable thickness, 1½ in. and upward, anchor pins are used, spaced about four to a square foot of surface. This is particularly effective where the surface is perpendicular.

In some cases our experience has been that about the only successful way to do this kind of work is to cut out the old work, dovetailing it and putting in anchor bolts. Care should be taken to see that the old work is roughened up and dovetailed, care being taken to avoid feather edging. Also in doing this class of work care should be taken to see that the new concrete is not mixed too soft, as it should be of such consistency that it can be lightly rammed. In repairing concrete work, where netting is used on the outer surface, care should be taken to see that the old concrete is removed back of the netting, so as to get a bond with the netting. Ordinarily the methods of patching and repairing as outlined above have been successful.

ILLINOIS CENTRAL R.R.—Our practice in patching small surfaces is as follows: A mortar is made up with the same proportions of cement and sand as has been used in the concrete which is to be patched. This mortar is quite stiff and considerable pressure is used in applying it to the cavity, which is first cleaned and thoroughly wet. After the mortar has partially set the surplus is rubbed off with a wooden float, leaving the surface of the patch flush with the surface of the wall. To lighten the patch and make it as near the color of the old work as possible, a small quantity of lime paste is added to the mortar. Where there are deep cavities, it is sometimes necessary to partly fill them and after the mortar has set make a second application.

CHICAGO & NORTHWESTERN RY.—In several cases, horizontal surfaces (especially bridge seats) have been damaged by frost, to a depth of 4 to 8 in. After removing the frozen and spoiled concrete, we chipped the surface down to a nearly uniform level, equal to the deepest frozen spot. In one case, the entire bridge seat of a three-track bridge had been severely damaged by frost. We placed dowel pins in squares of 2 ft., securely anchoring them in the sound concrete, then carefully and thoroughly cleaned and washed off the entire surface and placed new concrete.

During the past three years, we have successfully jacketed with concrete, old masonry which had badly deteriorated. The method pursued was the use of hook dowel pins and expanded metal, the jackets varying from 10 to 14 in. thick. We took great care to remove all deteriorated and loose portions of masonry, cleaning out joints, etc., and have been entirely successful in securing a perfect adhesion of concrete jackets to the old masonry; so much so, indeed, that we are gradually treating all our more or less deteriorated masonry of bridges on the Iowa Division in this manner. We have, unfortunately, a great deal of poor stone in bridge masonry. Some years ago there had been a great many back walls of bridges built up with 4 to 8 in. of concrete on top of old masonry, on account of track being raised by ballasting. In all of these cases no dowels were used and the method proved a failure.

MISSISSIPPI RIVER & BONNE TERRE RY.—In building a deck girder bridge, the abutments were found to be 2 ft. too far apart. The abutments were both on solid rock, so there was no danger from uneven settlement and a 2-ft. slab was patched on as follows: The face of one of the abutments was scabbled until thoroughly roughened. Holes 1½ to 2 in. diameter were drilled in the roughened face 12 to 18 in. deep and an area to a hole of 1½ to 2 sq.ft. They were not drilled in rows or to line, but irregular and promiscuously. Bolts of 1¼-in. iron with swelled end and split, with wedges, were driven into these holes, not tight enough to damage the old concrete, but firmly enough so they could not be pulled out. These bolts were carefully grouted, leaving the ends sticking out 12 to 22 in. The form was then built and concrete of the same proportions as the abutments was filled in and in course of a couple of months the girder was put in place. The end of the girder, 64 ft. long, rested on this patch which on one end of abutment is about 7 ft. high and other end 10 ft. or more. The work has been done between eight and nine years, and no sign of separation has shown.

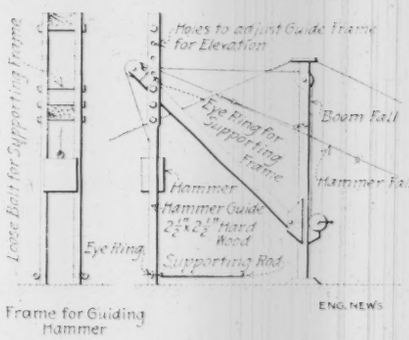
CHICAGO, BURLINGTON & QUINCY R.R.—The old concrete is thoroughly cleaned and wetted and about ½ in. of mortar put on. When this has set enough to become stiff the surface is roughened with the point of a trowel and another layer of similar thickness put on; this is continued until the hole is filled, the surface being finished flush with the old concrete. We have patched holes 3 and 4 in. deep and with an area of 2 or 3 sq.ft. successfully. Some of this work has stood for three years without signs of trouble. We have not succeeded in making patches less than 1½ in. thick stick, as the mortar cracks and scales off.

PHILADELPHIA & READING RY.—The first thing done is to scrape or pick off thoroughly all loose portions of concrete. The damaged face is then roughened up and holes 6 in. deep drilled therein, spaced about 2 ft. apart. The diameter of each hole is roughly 4 in. and the outline is made irregular. Enough of the old work is removed so that the new coat will have a minimum depth of 1 in. Along the perimeter of the damaged surface, the concrete is cut out to a depth of 2 in. with a slight overhang so that the old concrete will tend to lock in the new. The damaged surface is then brushed with a 1:1 cement wash.

Commencing at the extreme lowest point of the work the form is then built up 1 ft. at a time so that each filling can be thoroughly tamped. A mixture of 1 cement to 2 fine sharp sand is used in the actual work of restoring the surface. After the material has set and the form been removed, the surface is again wetted and treated to a 1:2 cement wash. This is a refinement, however, and is only employed where the maximum of neat finish is required. For ordinary railway work, we have not found it necessary except in cases where the boards used in the forms were rough and warped.

An Improvised Hand-Power Pile Driver

The pile-driver shown in the accompanying figure is a simply made frame which may readily be attached to any derrick and used to operate by hand power, if necessary, a hammer weighing from



A DEVICE FOR DRIVING LIGHT SHEET-PILING

300 to 500 lb. For sea-wall construction, for instance, it has been found very effective in driving the light sheet piles, particularly as it can be easily detached from the derrick, leaving that immediately available for use in lifting the rubble masonry into position. In ordinary soft earth requiring piles, sheeting up to 20 ft. in length may be driven with this ma-

chine by hand power. We are indebted to G. L. Bilderbeck, Civil Engineer and Supt. of Construction, U. S. War Dept., New London, Conn., for the drawing from which our cut was made.

Stresses in Rivets of Standard Beam-Connection Angles

By Jacob Friedland*

Recently the writer had occasion to design connections for beams spanning between long and shallow girders. These girders were subject to considerable relative deflection, and an investigation showed that it was unsafe to use any of the standard connecting angles given in the several handbooks. The investigation also brought out some interesting results with regard to the stresses in the rivets of the same connecting angles. These results, together with the method of arriving at them, are here presented.

For brevity, the legs of the connecting angles adjacent to the beam will be called the "beam legs"; the others are called the "girder legs." The rivets are called the "beam rivets," and the "girder rivets," respectively.

The usual method of procedure is to design the beam in bending, considered as simply supported, and to use where feasible the standard angles for connecting with girders. Attachment by angles introduces negative moments at the points of support. But as the maximum negative moment for a beam rigidly fixed at the ends is $\frac{pl^2}{12}$ for a uniform load, and $\frac{Pl}{8}$ for a concentration in the middle—less than the figured $\frac{pl^2}{8}$ and $\frac{Pl}{4}$ respectively—the beam is safe in bending. As a matter of fact, the ends are not rigidly fixed, and hence the real maximum bending in the beam lies between the given values.

The actual negative bending moment at the connection is reduced by the flexibility of the connecting angles, and is resisted by the moment couple made up of the pressure of the lower part of the angles against the girder web, and the pull on the rivet heads of the girder rivets above a certain point. These horizontal stresses are, in turn, transmitted to the beam rivets as shears on the latter. These stresses, as will be shown, are dangerously high in the usual connections, lying near the elastic limit of steel, and certainly very much above the unit-stresses allowed by any specifications. The direct pull on the girder-rivet heads is, in fact, prohibited by all specifications.

The notation used refers to Figs. 1 and 2:

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a = Distance between rivets along gage line;

c = Distance between gage line of beam rivets and back to angles;

d = Distance from the bottom of angles to a line above which no compression takes place, i.e., to the neutral axis (called N.A. in what follows), of the connecting system;

x = Distance from N.A. to the first girder rivet taking tension;

t = Thickness of the connection angle;

$\frac{K}{t}$ = Extreme-fiber stress in compression on the lowest part of the angles;

r = Pull on the first rivet above the N.A.;

B = Angle between the upper part of the back of the distorted angle and a vertical line = the inclination of the end of the beam below a horizontal line;

i = Moment of inertia of a section of length a of the outstanding leg $= \frac{at^3}{12}$;

I = Moment of inertia of the beam;

δ = Horizontal distance that the point F, opposite the first tension rivet, moves, when the angle is distorted.

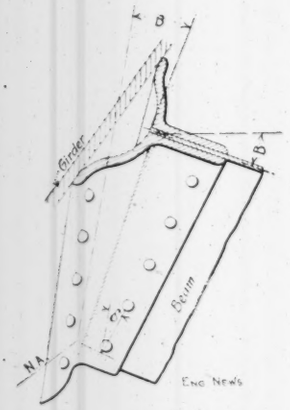

Fig. 1. Shape of Beam-Connection Angles as Distorted by Bending of Beam (Exaggerated)

The assumed mode of distortion of the connecting angles is shown in Fig. 1. The tension above the N.A., taken by the girder rivets only, stretches them, and also bends the girder leg, which acts like the half of a beam fixed at the ends with a concentrated load r at the middle. This is due to the fact that the outstanding leg of the angle is forced to lie flat against the web of the girder by the grip of the rivet.

The horizontal deflections of points on the gage line from their undeflected positions vary as their distances from the N.A. Each connecting angle is divided above the N.A. into as many horizontal strips of breadth a as there are girder rivets, and the stress and strain in each of these strips is considered uniform.

The deflection δ on the tension side consists of (1) an elongation of the rivet, assumed 1¼ in. long and ¾ in diameter $= \frac{5r}{0.6 \times 8 \times E}$; (2) the deflection of the girder leg of the connection angle $= \frac{2r}{3Et} \times \left(\frac{c}{2}\right)^3$; and (3) the elongation of the beam leg between the beam rivet and the back of the angle $= \frac{rc}{atE}$.

Below the N.A. the compression acts over the length d, the intensity increasing with the distance below the N.A. The deflection at the bottom of the angles $= \frac{Kc}{tE}$.

The N.A., i.e. the line above which the girder-rivet heads tend to be pulled off, is located as in the case of a reinforced-concrete beam, by the conditions that the deflection of any point on the end of the beam from its undeflected position varies as the distance of that point from the N.A., and that the sum of the tensions on the rivet heads equals the total compression on the beam legs below the N.A. Hence,

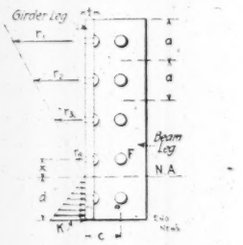

Fig. 2. Diagram of Moment Forces at Back of Beam-Connection

$$\frac{\delta}{x} = \frac{Kc}{dtE} = \frac{r}{Ex}\left[2\left(\frac{c}{2}\right)^3 \times \frac{1}{3t} + \frac{5}{8 \times 0.6} + \frac{c}{at}\right] \quad (1)$$

and,

$$\frac{dK}{2} = \frac{r}{x}[x + (x+a) + \ldots + (x+(n-1)a)] \quad (2)$$

where n = the number of rivets in tension.

These equations give the values of $\frac{K}{t}$ and x, which are independent of the actual value of the end moment.

The actual value of r is obtained from the end moment, and that in turn depends upon the length and moment of inertia of the beam and upon the method of loading. The case of a uniformly loaded beam is assumed in what follows, but central and other loads might be assumed. In fact the subsequent analysis is distinct from what precedes. But in order

TABLE OF RIVET STRESSES IN STANDARD BEAM-CONNECTION ANGLES
Computed for loads causing 14,000 lb. per sq.in. fiber stress in beam, for various spans

Beam	Connection angle	No. of rivets in each leg	Active rivets in girder	d, in.	l, ft.	Load, lb. per lin.ft.	End reaction, lb.	$\frac{pl^2}{12}$	Fixture moment, in.-lb.	Eccent. moment, in.-lb.	Moment on girder rivets, in.-lb.	Stresses in girder rivets; pulls on heads, lb. per rivet r_1	r_2	r_3	r_4	r_5	Moment on beam rivets, in.-lb.	Stresses in beam rivets, double shear or bearing r_1	r_2	r_3
24" I 80 lb.	4x4x⅞ 18 in. long	6	5	2.62	12	11,280	67,800	1,623,000	536,000	152,000	688,000	10,200	7,430	5650	3580	1380	384,000	18,200	10,900	3510
					20	4,050	40,500	1,623,000	730,000	91,100	821,100	12,200	9,520	6890	4300	1650	638,900	30,500	18,300	6100
					36	1,252	22,500	1,623,000	968,000	50,600	1,018,600	15,100	11,800	8520	5300	2040	917,400	44,900	26,800	8960
20" I 65 lb.	4x4x⅞ 15 in. long	5	4	2.195	12	7,560	45,400	1,092,000	367,500	102,000	469,500	9,800	7,200	4590	2000		265,500	17,850	8,925	
					18	3,375	30,300	1,092,000	471,000	68,100	539,100	11,200	8,250	5280	2290		402,900	27,000	13,500	
					24	1,890	22,600	1,092,000	549,000	51,000	600,000	12,520	9,200	5890	2550		498,000	33,200	16,600	
18" I 55 lb.	4x4x⅝ 15 in. long	5	4	2.195	12	6,390	38,350	940,000	404,500	86,000	490,500	9,360	6,890	4400	1910		318,500	21,100	10,550	
					18	2,550	22,900	940,000	499,000	51,500	550,500	11,410	8,410	5380	2350		417,500	29,900	14,900	
					24	1,432	17,200	940,000	564,000	38,600	602,600	12,520	9,200	5900	2550		525,400	35,000	17,500	
15" I 42 lb.	6x4x⅞ 10 in. long	4*	3	1.45	12	3810	22,900	550,000	166,500	77,200	243,700	8,710	5,700	2,756			89,300	5,800	1,780	
					20	1,375	13,750	550,000	220,000	46,400	266,400	13,260	6,670	3200			173,600	11,300	3,450	

*Six rivets in the beam leg.

to show what stresses are generated in the rivets under the conditions recommended in the steel handbooks, uniformly loaded beams, loaded to a fiber stress of 14,000 lb. per sq.in. if simply supported, have been taken. Using the familiar textbook notation we have for the equation of the elastic curve of such a beam

$$EI\frac{d^2y}{dz^2} = M + \frac{p}{2}(lz - z^2)$$

where M is the end moment, unknown as yet. By integration we get

$$EI\frac{dy}{dz} = Mz + \frac{p}{2}\left(\frac{lz^2}{2} - \frac{z^3}{3}\right) + c$$

When $z = 0$, i.e., at the connection angle,

$$EI\frac{dy}{dz} = +c = -\frac{\delta}{x}EI$$

At the center of the span l, the beam is horizontal, hence

$$\frac{dy}{dz} = 0 = \frac{1}{EI}\left[\frac{Ml}{2} + \frac{p}{2}\left(\frac{l^3}{8} - \frac{l^3}{24}\right)\right] - \frac{\delta}{x}$$

or

$$M = \frac{2\delta}{x} \times \frac{EI}{l} - \frac{pl^2}{12} \qquad (3)$$

Now, $-\frac{pl^2}{12}$ is the end moment for a beam fixed at the end, and uniformly loaded, and the equation shows that this moment is diminished by $\frac{2\delta}{x} \times \frac{EI}{l}$, a quantity which varies directly with the flexibility of the angle $\frac{\delta}{x}$ and the rigidity of the beam EI, and inversely with the length of the beam. In other words, for stiff, short beams, and light connecting angles in which the gage of the rivets between the outstanding legs is great, the decrease in the end moment is great.

The bending moment of the forces on the connecting angles is given by

$$M = -2\left[\frac{r}{x}\left\{x^2 + (x+a)^2 + \ldots + (x+(n-1)a)^2\right\} + \frac{Kd^2}{3}\right] \qquad (4)$$

Twice the bracket is taken, since there are two angles with two sets of rivets, while there is only one beam upon which they act. The moment is negative. From (3) and (4) we get

$$\frac{2\delta}{x}\frac{EI}{l} - \frac{pl^2}{12} = -2\left[\frac{r}{x}\left\{x^2 + (x+a)^2 + \ldots + (x+(n-1)a)^2\right\} + \frac{Kd^2}{3}\right] \qquad (5)$$

Since $\frac{\delta}{x}$, x and $\frac{K}{r}$ are known from equations (1) and (2), the substitution of these values in (5) gives the value of r, i.e., the pull on the first girder rivet above the N.A. The stress in the other rivets varies as their distances from the N.A.

The moment M, found above, also produces shearing stresses in the beam rivets. For the latter the N.A. is located through the center of gravity of the group of beam rivets, and the stresses are found on the assumptions that the rivet shear varies as the radial distance of the rivet from the center of gravity of the group and is perpendicular to that radius, and that the sum of the moments of the rivet shears about the N.A. is equal to the moment at the end of the beam.

In addition to the bending moment which comes from the partial fixedness of the beam end, and which is therefore called "fixture moment," there is a moment on the connecting system due to the fact that the end reaction has a lever arm equal to the distance between the centers of gravity of the groups of beam rivets and girder rivets respectively. This is called "eccentricity moment," and is to be added to the moment which tends to pull the heads off the girder rivets, and is to be deducted from the moment which tends to shear the beam rivets.

The stresses found from the moments mentioned are to be combined, vectorially where necessary, with the actual shearing forces as ordinarily computed. The latter have been entirely neglected in what follows, and may be added by anybody who thinks the rivets have not enough work to do to stand up under the moment stresses.

In the preceding analysis no account has been taken of the distortion or yielding of the girder web; hence it applies strictly only to the connections of similar beams which are framed opposite each other and loaded so as to produce equal end moments at the connections. It will also apply to beams which are framed into rigid columns, or into girders stiffened against lateral distortion. Even in the case of beams framed only on one side of a girder, the direct pull on the rivet heads is excessive.

EXAMPLE—A 20-in. I-beam, 65 lb., 18 ft. long, loaded with 3375 lb. per lin.ft., and having two 4x4x⅞-in. connecting angles 15 in. long, with five ¾-in. rivets in each leg (upset to ⅞ in.) and 1¼ in. long in the girder legs—all as given in the "Carnegie Handbook"—will be worked out. Here, $a = 3$ in., $c = 2\frac{1}{2}$ in., $t = \frac{7}{8}$ in., $I = 1169.6$. The solution of equations (1) and (2) gives $x = 2.305$ in. below the second rivet from the bottom, $\frac{K}{r} = 10.92$, $\delta E = 65.76r$. Substituting these values in equation (5), we get,

$$2\frac{r \times 65.76}{2.305} \cdot \frac{1169.6}{18 \times 12} - \frac{3375 \times 18 \times 18 \times 12}{12} = -2\left[\frac{r}{2.305}(2.305^2 + 5.305^2 + 8.305^2 + 11.305^2) + \frac{10.92 \times 2.195^2 r}{3}\right]$$

or

$$r_4 = 2015 \text{ lb.},$$
$$r_3 = 4630 \text{ lb.},$$
$$r_2 = 7250 \text{ lb.},$$
$$r_1 = 9850 \text{ lb.},$$

These are pulls on the rivet heads, somewhat eccentric.

The end moment has been reduced from $\frac{pl^2}{12} = 1{,}092{,}000$ in.lb. to 470,000 in.lb., by the flexibility of the connection.

The reaction of 30,300 lb., with a lever-arm of about 2¼ in., gives an eccentricity moment of 68,100 in.lb., or about 14% additional, making the rivet stresses in the girder leg 11,300, 8300, 5300 and 2300 lb., respectively.

The center of gravity of the beam rivets passes through the middle rivet of the group, which is not stressed by the moments. The total bending to be taken care of by them = 470,000 − 68,000 in.lb. = 402,000 in.lb., which by the method outlined above produces stresses in the outermost and the adjacent rivet of 27,000 lb. and 13,700 lb., respectively —a double shear or a bearing stress, but evidently large enough for both and "some to spare," according to specifications. The direct shear, acting at right angles to the moment stresses in this case, is to be combined vectorially with the above.

In the accompanying table are given the results of computations for Carnegie standard connections, and for loads which

would produce a fiber stress of 14,000 lb. per sq.in. in simply supported beams. Some of the Cambria connections have been worked out, and the stresses are of about the same magnitude. It is to be noted that the rivets are ¾ in. diameter, and that the stresses are per rivet, and not per square inch of metal.

The writer has not been very successful in getting information about tests on connecting angles. The articles on p. 318, Vol. I, 1895, and on pp. 43 and 203, Vol. II, 1898, ENG. NEWS, to which he has been referred, deal only with simple connections for the smallest beams, having one girder rivet and two beam rivets, and concern themselves with the effects of the eccentricity moment, i.e., with the least important of the two end moments.

The method of failure of a steel-frame building in St. Louis, destroyed by a cyclone, and described in Vol. XXXVII of the *Transactions* of the American Society of Civil Engineers, is worthless as a criterion for stresses. But the description contains a picture of a connecting angle, with some of the rivets that failed, which seems to justify the assumption of the distortion which a connecting angle suffers under stress.

Until the correctness or incorrectness of the stresses in the accompanying table is established by tests it would seem proper to use some other form of connection for beams where the maximum loads are quickly and frequently applied, i.e., some form of shelf or hinge, which does away with the fixture moment.

In making tests on connecting angles it is not necessary to use full-size members. The correctness of the assumptions can be demonstrated by careful micrometer work on smaller angles and beams, having an arrangement of rivets similar to that in full-size members. The simplest test consists in loading the end of an I-beam, cantilevered out from some rigid support to which it is connected by a standard angle.

If the advantage of stiffness of structure resulting from rigid connections is desired, the stresses should be calculated and a proper form of connection should be used.

Instructions to Young Engineers on Highway and Bridge Work

In its work of building roads and highway bridges, the State Highway Commission of Illinois employs a number of young engineers, and as these positions are under civil service, it is possible to build up and develop a staff of men experienced in this line of work. For the use of these assistants a manual of instructions to engineers has been issued recently by the Highway Commission (Arthur N. Johnson, State Highway Engineer, Springfield, Ill.). Extracts from the general instructions may prove suggestive to others:

While in the employ of the Illinois Highway Commission you are expected to accomplish two things: 1, well executed work, and 2, the coöperation and support of the people of the community in which the work is situated. One is as important as the other. You must bear in mind that your actions will govern the opinion the local people will form, not only of the piece of work in hand, but of the whole conduct of the department. You should, therefore, see to it that this opinion is favorable in so far as strict attention to your work, and a business-like, courteous manner on your part can form such an impression.

Always give courteous attention and consideration to all inquiries for information and should questions be asked on any point about which your instructions forbid your giving such information, explain the situation, stating that you will be glad to refer the question to the office if so desired. If questions are asked you on matters about which you are in doubt, say so frankly, but add that you think you will be able to secure the information.

If the plan of the work or the methods required by the specifications are at all in question, you may explain that these are a part of your instructions which you have no authority to change, and moreover, that they are based on wide experience in such work and may confidently be expected to produce good results.

If at any time verbal instructions are given you by any engineer in charge of work, which modify any written instructions that you may have at hand, or the specifications and plans as they are already drawn up, demand of the engineer giving you such instructions that he make them in writing, and you should immediately forward a copy of these instructions to the office. Otherwise, if you make any change in the plans or your written orders, you will be considered responsible and held as having violated your instructions.

You should make no estimates in the field. If estimates are called for, you should state that it is a practice of the office that all such be checked before they are given out, and that your instructions require that you forward the data you have in hand to the office.

In your dealings with contractors, keep in mind the fact you are on the work to see that it is done and materials furnished in accordance with the specifications. Usually the specifications will describe the result desired. If this is the case, then the contractor is at liberty to exercise his judgment as to the methods he will pursue. If you are convinced such methods will not produce the results desired, you may so inform the contractor and advise the office immediately.

You doubtless will come across contractors more or less inexperienced with the particular kind of work in hand, and if they care to have your advice in the matter, you may give them such suggestions as you may know will give satisfactory results in an economical manner.

It is an unfortunate fact that you will find a few contractors who will endeavor to see just how far from the specifications they can go, and it is well to make it plain at the beginning that nothing but good work will be accepted. On the other hand, you will find the average contractor desires to do a creditable piece of work, and under these circumstances, if you do your work properly, there will be no question as to the results.

You should be no more exacting of a contractor because he will secure a profit from his work than if he has undertaken the work at a loss; but the fact that he is doing the work at a loss does not justify you in permitting any inferior work or materials.

Topographical Surveys for a Large Irrigation Project in Southern Alberta

BY PERCY SCHÜTTE*

The 400,000-acre tract of the Southern Alberta Land Co., Ltd., lies between Calgary and Medicine Hat and is adjacent to and south of the great 3,000,000-acre tract of the Canadian Pacific Ry. For the most part it is composed of a rolling prairie, but the rivers are from 200 to 300 ft. below the bench lands and are bounded on either side by steep banks, interrupted occasionally by a mile or two of river flats.

The tract is divided naturally into three districts, as shown on the accompanying map; that lying to the immediate west of the Bow River, that lying to the immediate east of the river, and that surrounding the company's townsite of Suffield. To irrigate these lands the company is taking water from the Bow River, at a point about 45 miles below Calgary, and bringing it 120 miles, by means of ditches, flumes and inverted siphons. In addition to this area there is the tract of the Alberta Land Co., containing 75,000 acres, which is to be irrigated by the Southern Alberta Land Co., Ltd., canals.

The extraordinary length of the main canal is due to the very easy grade of the Bow River. In getting away from the river at the intake of the canal considerable difficulty was experienced, and it was finally accomplished by means of a 6000-ft. cut having a maximum depth of 65 ft.

Topographical surveys of the lands of the two Alberta land companies above mentioned, comprising together nearly half a million acres, are now being made, irrespective of the possibility of irrigating the whole of them by means of the gravity system. It is probable that schemes for pumping water to the higher levels will be considered later, when these surveys will prove invaluable in deciding the feasibility of such schemes.

The government surveys have already divided the land in Alberta into townships and the townships into sections one mile square. These lines are marked at the northeast and quarter-east corners of every section, and at the quarter-north corner of every section which has a road allowance to the north of it, by mounds surrounded by four 2-ft. square pits at

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the corners of a 6-ft. square. Road allowances, 66 ft. wide, occur once every mile from east to west, and once every two miles from south to north.

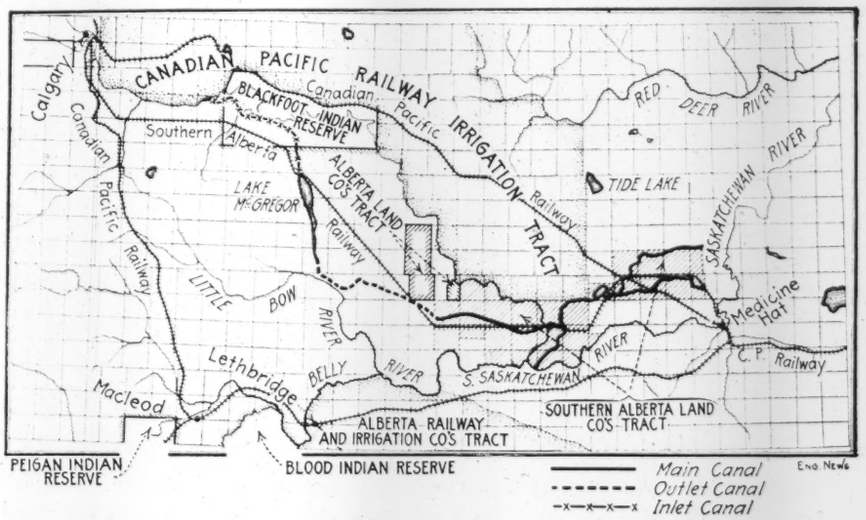
Before the taking of topography was begun, a level party obtained elevations on all the section and quarter-section corners. These elevations were referred to the datum of the ditch elevations, so that in every district the ditch and topographical elevations coincide. In leveling no error on any circuit amounting to more than 0.03 ft. per mile and varying as the square root of the total mileage was allowed, and any error less than that, which did occur, was spread proportionately over the entire circuit.

Plane-tables are used for taking topography. The instruments consist of an 18x24-in. board fitted with rollers on which any convenient length of paper can be placed, and the usual alidade with fixed stadia hairs.

Stadia rods 15 ft. long and of special design were used. The graduations for the foot marks are the centers of the figures 1, 2, 3, etc., V being used for 5, N for 9 and X for 10; beyond X, 1, 2, 3, etc., are repeated. The half-foot graduations are designated by two fine-pointed wedge-shaped marks, broken in the middle. The only other marks on the rod are heavy line squares placed with the diagonals horizontal, and so arranged on the rod that the diagonals on the 0.3 and 0.7 graduations. The vertical diagonal is 0.2 long, thus leaving 0.1 between the top of the square and the top or bottom of the figure representing the foot mark, which is made exactly 0.2 high. The graduations are thus 0.1 apart and very plain. On clear days it has been possible to get accurate readings at a distance of 2800 ft.

Two rodmen are employed for each plane-table, and in country where it is not necessary to read many vertical angles, the topographer is easily able to keep both busy by taking readings at least once in every 250 ft. Readings are, of course, taken wherever an irregularity in the ground occurs and not at any stated interval; but it has been found in practice that it is generally necessary to take a reading at least once in 250 ft., even where the ground has a fairly uniform slope, if reliable topography is required.

Each party consists of a topographer, a recorder, two rodmen and a teamster. The recorder is equipped with a stadia slide-rule, with which he obtains the difference in elevation when the vertical angle is read to him by the topographer. When it is necessary to move the position of the plane-table, the head rodman sets a station ahead in a suitable place, and the other rodman moves to the old setup to give a backsight. The average number of stations necessary for one section is about 8, and a good day's work is about 400 acres.



MAP SHOWING THE IRRIGATION PROJECT OF THE SOUTHERN ALBERTA LAND CO., ALBERTA, CANADA

The scale used on the plane-table map is 1 in. = 400 ft., which was found to be the smallest scale to which reliable topography could be taken. This scale gives a map for each section which is a little more than 1 ft. square, and makes it possible to plot contours to 1-ft. or 5-ft. intervals, as the nature of the ground requires.

When these field maps are completed they are turned over to the camp draftsman, who puts them in shape for filing. They are then sent to headquarters, where they are traced. It is the intention of the company to provide each purchaser of a piece of land with a blueprint map of his section, and to indicate on these maps the way in which the piece of land can be irrigated to the best advantage.

When the topography of each township is completed a map is made of it on a reduced scale of 1 in. = 1000 ft., which is used for laying out a water-distribution system. Finally, maps to a scale of 1 in. = 2500 ft. will be made of the various districts, which will be used for a similar purpose.

The estimated cost of the topographical survey of the entire tract, including the lands of the Alberta Land Co., is $35,000.

A New Stadia Attachment and Diagram

By R. L. McCormick*

This instrument, which can be attached easily to any transit or plane-table alidade, obtains directly the elevation and horizontal distance of a point observed by the stadia method of surveying. It dispenses with the reading of the vertical angle and thus eliminates the usual tedious reduction by means of a stadia slide-rule or other devices, and it also eliminates the subtraction or addition of the vertical distance from the elevation of the transit station.

These results are accomplished in the following manner: The elevation of the transit station is set on scale 3 (Fig. 1) by means of vernier 7 previous to taking the observations. The stadia reading is set on scale 1 by vernier 5 and scale 2 is brought into contact with the rotating vernier 6. This vernier reading is the elevation of the observed point and the reading of vernier 8 on scale 4 added to half the stadia reading will be the horizontal distance.

Vernier 5 reads to 1 ft., verniers 6 and 7 to 0.1 ft., and vernier 8 to 0.5 ft. An adjustable stop is provided at 9 to bring the instrument to zero reading with telescope horizontal. The constant of the transit or plane-table telescope has not been included in the instrument shown in

*Professor of Mechanics, Rose Polytechnic Institute, Terre Haute, Ind.

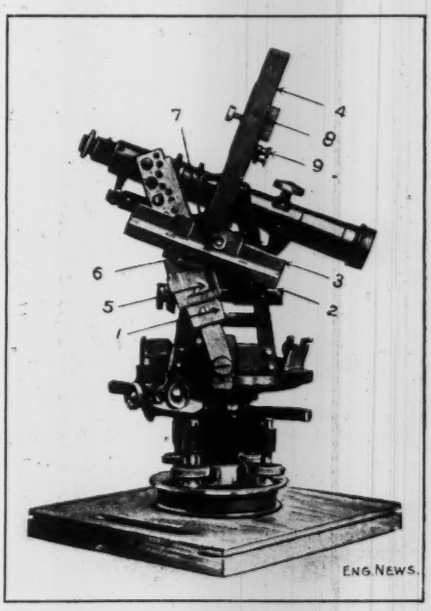


FIG. 1. MCCORMICK STADIA ATTACHMENT APPLIED TO A SMALL TRANSIT

Fig. 1, but will be provided for in the construction of future attachments, as will be demonstrated later in this article.

The mathematical principles involved in the device are as follows: A, in Fig. 2, is in the horizontal axis of the telescope; B moves along the line AB as arm AC revolves about point A. If arm AC,

FIG. 2. DIAGRAM OF PRINCIPLE OF MCCORMICK ATTACHMENT

when in line AB, is clamped to the telescope axis with the telescope horizontal, then angle BAC is equal to the vertical angle a of the transit and, since $AC = BC$, angle $ACE = 2a$. Now if $CE = \frac{R}{2}$ where R is the stadia reading, then $DE = \frac{R}{2} \sin 2a$ and $CD = \frac{R}{2} \cos 2a$.

Since the vertical distance is $R \sin a \cos a + f$, $\sin a = \frac{R}{2} \sin 2a + f \sin a$, and horizontal distance is $R \cos^2 a + f \cos a = \frac{R}{2} (1 + \cos 2a) + f \cos a$, it follows that ED is the vertical distance and $CD + \frac{R}{2}$ is the horizontal distance of the observed point, neglecting the terms $f \sin a$ and $f \cos a$.

The scale of CE is half that of CD or DE, so that the stadia reading R when set on the scale CE is actually $\frac{R}{2}$ on the scales CD or DE.

The quantity $f \sin a$ is practically equal to $\frac{f}{2} \sin 2a$ within the limits of the usual vertical angles; and, by setting the zero of scale CE, f feet measured on scale CE (which is $\frac{f}{2}$ on scale ED) from C toward E, this quantity is included in the operation.

The term $f \cos a$ may be included also by a proper setting of the zero on scale CD, but this is an unnecessary refinement as $f \cos a$ is practically f and can be simply added to each horizontal distance. In plotting the notes obtained by this device a scale having the graduations on half of its length double those of the other would be found desirable, as the $\frac{R}{2}$ could be added to the $\frac{R}{2} \cos 2a$ mechanically. The larger scale would evidently be the scale of the map and R would be halved by reading R on the smaller scale.

The writer originated this instrument about ten years ago and tried then to double the vertical angle by means of two gears. But the necessary accuracy of these gears could not be obtained; $^1/_{1000}$ in. lost motion would make a difference of 1 min. of arc here unless the gears were quite large, which, of course, would be cumbersome in such a device. One or two other schemes were also abandoned for similar reasons.

The present arrangement, which is extremely simple, was devised a few months ago. The writer made some computations to determine the limits of error in the lengths of arms CA and CB, Fig. 2, as well as in the deviation of the slide B from the straight line BA.

The result of these computations showed that to change angle $ECA = 2a$ into $2a \pm 2$ min., which is equivalent to 1 min. error in a, would require a difference in the length of the arms CA and CB varying inversely as the angle a for small values of a, of 0.1 in. for $a = 1°$ and 0.01 in. for $a = 10°$. This is a decided advantage in the making of the attachment, as the manufacturer is allowed greater leeway. It also keeps the instrument accurate in spite of wear, as a larger part of the angles in stadia surveying are less than 10°. The slide B could deviate $^1/_{200}$ in. in 1 in. of travel, from the straight line AB to produce an error of 1 min. in a. This would amount to $^1/_{50}$ in. at A, as the distance AB is about 6 in., and could easily be taken care of by the instrument maker. The arms CA and CB are about 3 in. long.

The attachment, excepting the slides and graduations, was made by a machinist in the shops of the Rose Polytechnic Institute and has been tested by numerous observations checking with the ordinary stadia method with practically no error except those resulting from the personal equation.

As shown, it has a range of 550 ft. in stadia reading and 400 ft. in vertical distance, but this range can be increased by some slight modifications without increasing the size. The weight is 22 oz., but this can easily be reduced to 10 or 12 oz. and the device still have the requisite rigidity as there are at least 8 to 10 oz. of superfluous metal in this attachment.

The time of a stadia observation with this attachment is about 10 sec. greater than by the usual method, but, as the final result is obtained in the field by means of this instrument, I estimate the saving of at least one man in stadia surveying as well as the elimination of several chances of error in each observation.

The fact that final results are not obtained in the field is, from my experience as a teacher of surveying as well as in practical work, the principal reason why stadia surveying is not more commonly used. Especially is this true of students: the chances of error are so many and the checks upon error so few that they are not inclined to use this method in their future surveys. These remarks, of course, do not apply to men

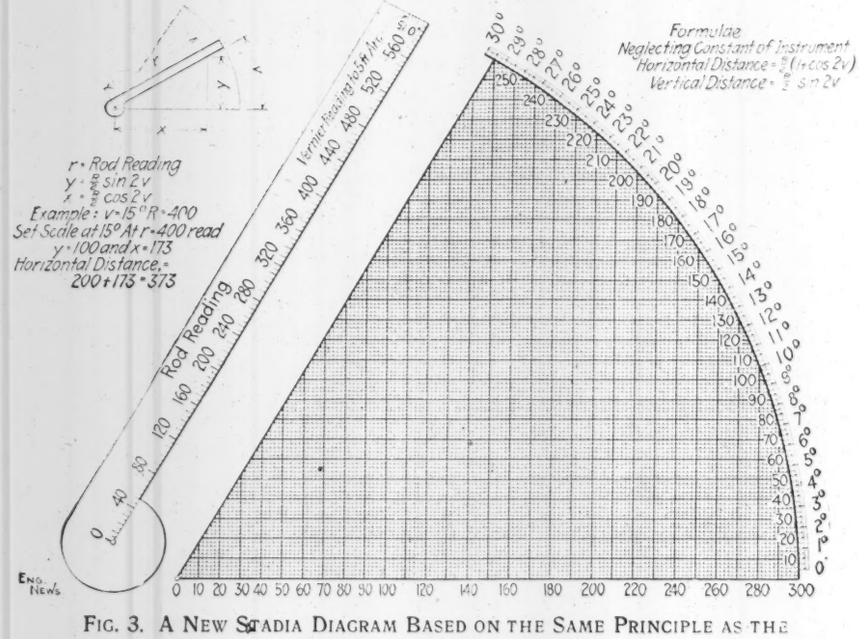

r = Rod Reading
y = R/2 sin 2v
x = R/2 cos 2v
Example: v = 15° R = 400
Set Scale at 15° At r = 400 read
y = 100 and x = 173
Horizontal Distance = 200 + 173 = 373
Vertical Reading to 5 m/m
Rod Reading
Formulae
Neglecting Constant of Instrument
Horizontal Distance = R/2 (1+cos 2v)
Vertical Distance = R/2 sin 2v
Eng. News

FIG. 3. A NEW STADIA DIAGRAM BASED ON THE SAME PRINCIPLE AS THE MCCORMICK ATTACHMENT

who have become expert in stadia surveying.

The attachment is readily removed, as can be easily seen from Fig. 1. The transit shown is smaller than the ones commonly used in stadia surveying.

Fig. 3 is a stadia diagram based upon the same principles as the stadia attachment and is self-explanatory. A range of 600 ft. in stadia reading and a vertical angle of 30° may be obtained on a diagram 13x15 in., reading vertical distances to 0.1 ft. and vertical angles to 1 min. The scale shown on the left is placed upon the diagram with its zero coincident with the zero of the diagram, and may be attached with an eyelet.

Patents have been applied for covering both the stadia attachment and the stadia diagram. Within my knowledge, the only instrument, other than this stadia attachment, for obtaining elevations direct by stadia reading is the Wagner-Fennel tachymeter, which is fully described in the U. S. Coast & Geodetic Survey Report for 1891 (appendix 16). This device differs in three essential features from the one shown in this article: (1) It does not double the vertical angle; (2) all scales are graduated alike, and (3) the stadia rod is held perpendicular to the line of sight. These differences are sufficient to show the novelty of this stadia attachment and the writer will be very grateful for any criticisms or suggestions that may tend to improve a device which he hopes may prove to be of some importance.

The stadia attachment, when made in the future, will probably be attached on the left side of the transit, but in the one shown, it was placed on the right side so that it could easily be calibrated by means of the vertical arc.

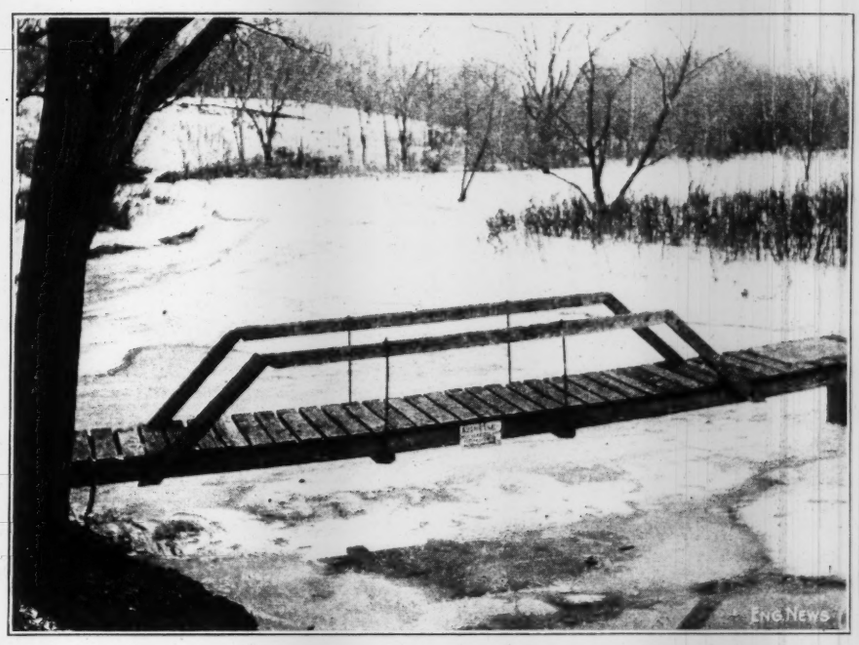
I am indebted to Prof. J. B. Peddle for suggestions and assistance in the design of this instrument and also in the preparation of the cuts for this article.

Notes

Bridge Jumps a Foot—A recent newspaper account reports unusual behavior on the part of a bridge:

Joseph Kuntz, proprietor of the Pine Tree Hotel, found that massive cakes of ice were being thrown against the steel bridge by the current. Kuntz managed to get on the structure, when a small wooden bridge, followed by hundreds of tons of ice, swept down the middle of the stream, striking the bridge, diving underneath and raising the ponderous structure over a foot. He expected the bridge to go, but it dropped back on its foundations.

The graphic term "ponderous structure" contrasts refreshingly with the terminology lately adopted in Kansas, where steel highway bridges have come to be called "tin bridges," to the delight of the promoters of concrete bridges. Students of impact questions will find in the above account some food for reflecting on the possible need of providing large impact allowances, if bridges are to resist the effects of being lifted a foot and dropped back.



FOOTBRIDGE OVER MINNEHAHA CREEK IN A MINNEAPOLIS PARKWAY; IT IS STILL STANDING

Laughing Water—A sign on the footbridge shown in the view bears the warning:

$25.00 FINE FOR MUTILATING THESE BRIDGES IN ANY MANNER

The designer of the structure is indebted $25 to some authority, since he mutilated it very effectively by leaving out the diagonals and by making the "truss" shorter than the span. The "bridge" is a reality, as the picture and the following letter prove:

This foot bridge is located in a Minneapolis parkway and spans Minnehaha Creek about four miles above the famous falls.

This is a mental test picture. Anyone totally devoid of a sense of the relation of form and function can look at it without smiling. Are you smiling?

E. F. PABODY, JR.

Minneapolis, Minn., Mar. 14, 1912.

Even the water, the "Laughing Water" Minnehaha, smiles at this bold bridge. Further information on the dimensions of the structure is as follows: Span, 24 ft. clear; width, 3 ft.; length between feet of "end posts," 15 ft.; depth of "truss," about 3 ft.; main timbers, each of two 2x6's, spiked together; brace timbers, 4x4; "hangers," ½-in. gas pipe; lateral bracing between main timbers, 2x4. The inclined 4x4 legs are connected to the main timbers by a lag-screw. The deflection of the bridge at the middle is 3½ ins.

A Beam Problem of some interest is submitted by E. C. Dilworth, 1422 Frick Bldg., Pittsburg, Penn. The conditions

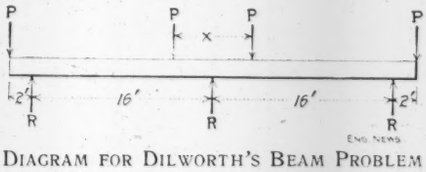
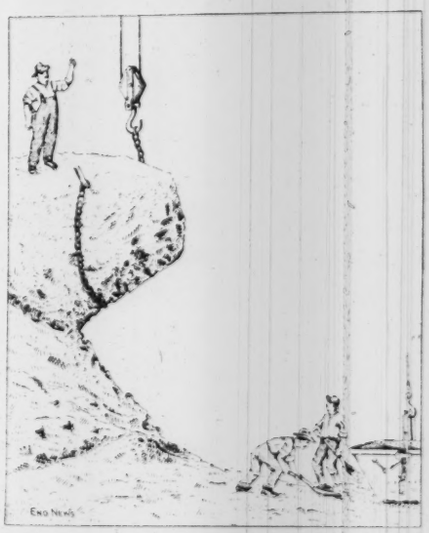


DIAGRAM FOR DILWORTH'S BEAM PROBLEM

are as indicated in the accompanying sketch. The four loads P are all equal and it is required to find the distance x to make the three reactions R equal. Mr. Dilworth states that "this, of course, could be solved by trial, but expressions for R and for x are interesting." We are unable to guess how these "interesting expressions" were obtained, but a straight solution by the three-moment theorem leads to a cubic equation for x, which gives a value of about 10.58 ft.

Excavating Earth with Derrick Chain—A correspondent describes the use of a novel method of earth moving, as follows:

At one of the subway excavations in New York City recently a large pile of excavated material which was being



kept for use as concrete aggregate had been frozen to quite a depth, so that as it was dug away at the bottom an overhanging knob was left above. To cut off this knob, a steel bar was driven into the root of the projecting mass from the top; the end of a derrick chain was fastened to the bar, and the chain was looped around under the knob. The derrick hook pulling on the chain sheared off the knob neatly.

A. N.

Brooklyn, N. Y., Mar. 19, 1912.

A Suggestion for Circular Verniers—An improvement in a detail of engineers' apparatus, the vernier, is described below. It pertains to verniers of circular scales only, where the scale is numbered in both directions and readings may have to be taken either to right or to left. As described, it was used on a protractor, but it would apply similarly to the circles of a transit:

Sir—A short time ago, having occasion to order a protractor for a work requiring a large number of settings, it occurred to me that greater ease of reading might be secured by separating the forward and backward reading verniers, thereby making the index line of the main scale more prominent than in the customary design where this line constitutes the zero point of both verniers. A protractor so equipped was found to answer well its intended purpose, and the slight modification is submitted herewith in the belief that it may have a wider sphere of application.

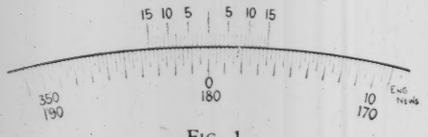


FIG. 1

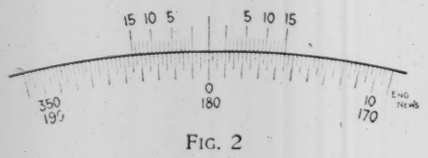


FIG. 2

The sketch, Fig. 1, represents the customary form of vernier. Fig. 2 is the same vernier with the zero lines separated an integral number of divisions, leaving a clear space about the index line. The number of divisions included in this space is, of course, immaterial. The zero lines of the verniers may be omitted, but it is better to retain them as they serve to check integral division settings of the main scale.

Very truly yours,
Lucien E. Picolet.
2217 N. 52d St., Philadelphia, Penn., Apr. 1, 1912.

The proposed arrangement may lead to mistakes by reading the division under the zero line of the vernier instead of under the index line. But the argument in favor of the arrangement is worth consideration.

The Problem in Track Re-alignment, published in our issue of Nov. 23, 1911, p. 628, is much simplified if the case is one where the two curves to be compounded are already joined by a simple reversed curve, and all the elements of all the curves are known. Mr. T. R. Howser, Resident Engineer of the Madeira-Mamore Ry., in Brazil, sends us a solution for such a case where solving two triangles gives sufficient data for finding the radius and central angle of the new curve. There are, of course, several ways in which the problem, when it occurs in location work and all the elements are readily obtainable, may be solved without recourse to field work, as Mr. Howser states; but we believe the method given in our issue of Nov. 23 has distinct advantages in re-aligning old track.

Solutions of the Printing Press Gear problem, proposed in our issue of Mar. 21 by H. M. Lloyd, have been received from James Clark, Bradford, Penn.; F. L. Griffin, Reed College, Portland, Ore.; Albert E. Guy, Pittsburg, Penn., and A. C. Hough, Munhall, Penn. No two of these, however, agreed in the form of equation for the unknown curve, although two or three of the equations were equivalent to each other and to the form given below.

The equation for this curve (F C G in the diagram) can be most readily obtained by taking advantage of the condition that, for pure rolling contact between two curves turning about fixed centers, the point of contact must be always on the line joining those centers. Then with the center A in any position, such as A_1 in the diagram, the point of contact will be at C_1 on the line A_1 B. Taking B as the origin for a polar equation for the curve C F, the radius vector for any point such as C_1 is r, and the angle is θ.

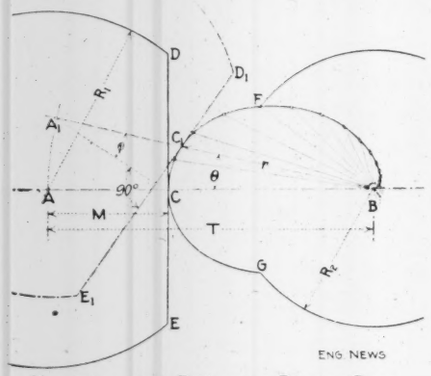


DIAGRAM FOR PRINTING-PRESS GEAR PROBLEM

The angle ϕ (see diagram) is seen to be equal to the angle between the tangent to the curve at C_1 and a normal to the radius vector r. This angle ϕ is, therefore,

$$\text{Tan}^{-1}\frac{dr}{r\,d\theta}$$

Now, letting T equal the distance A B and M equal the distance A C, an inspection of the diagram shows that

$$\frac{T-r}{M} = \sec\phi = \sec\left[\tan^{-1}\frac{dr}{r\,d\theta}\right]$$

$$= \sqrt{1+\left(\frac{dr}{r\,d\theta}\right)^2}$$

Clearing of radicals, transposing and extracting the square root reduces this to the following:

$$\frac{\frac{dr}{d\theta}}{r\sqrt{(T-r)^2-M^2}} = \frac{1}{M}$$

Integrating and transposing, etc., we obtain

$$re^{P} = \frac{T^2-M^2}{M}\,e^{\frac{T}{M}r+\sqrt{\frac{T^2}{M^2}-1}\sqrt{\frac{(T-r)^2}{M^2}-1}}$$

in which

$$P = (KM-\theta)\sqrt{\frac{T^2}{M^2}-1}$$

The constant of integration K is found to be zero, since $\theta = 0$ when $r = T - M$.

By making use of the relationship between e and the hyperbolic trigonometric functions, the equation between r and θ can be reduced to the form

$$r\left[\frac{T}{M}+\cosh\left(\theta\sqrt{\frac{T^2}{M^2}-1}\right)\right] = \frac{T^2-M^2}{M}$$

From this equation the curve can be plotted by assuming successive values for θ and computing the corresponding values of r, making use of a table of hyperbolic cosines. The curve C F B in the diagram was plotted in this way. It is a spiral around the origin B, toward which it approaches more and more closely as θ is increased.

One point which seemed to cause considerable unrest in the minds of those who submitted solutions was the difficulty of obtaining the values of the radii R_1 and R_2 of the two gears and the distances A C and C B, although all of these were specified as "given" quantities.

These quantities are connected by the condition that the total peripheries of the two gears must be the same. As actually constructed, the gears have two flat sides and two "bumps," respectively, at 180° apart. That is, they are symmetrical in the original position about vertical axes through the centers A and B. This was not mentioned in the original statement of the problem and has nothing to do with the equation of curve F C G.

Mr. Griffin suggests a solution for R_1 and R_2 by trial (assuming that the distances A C and C B are given) as follows: Take the various values of r, found from the equation for the curve C F B, for corresponding values of θ and use them as trial values for R_2; obtain R_1 by subtraction from T, and the angle D A C through its cosine which is $\frac{M}{R_1}$.

Then test the value of the expression

$$R_2\left(\frac{\pi}{2}-\theta\right) - R_1\left(\frac{\pi}{2}-\cos^{-1}\frac{M}{R_1}\right)$$

which would be exactly zero for a perfect solution.

The Problem in Street Location given by R. R. Cook, of Indianapolis, Ind., in our issue of Mar. 21, p. 541, has brought to hand several letters from engineers who differ with Mr. Cook as to the simplicity of the method given by him. The original problem is reproduced in Fig. 1.

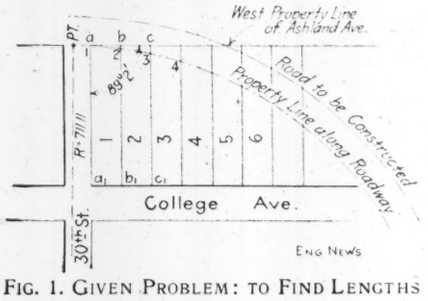


FIG. 1. GIVEN PROBLEM: TO FIND LENGTHS a1, b2, c3, ETC.

A simple trigonometrical solution is that sent by Geo. H. Harris, Division Engineer of the Michigan Central R.R., at St. Thomas, Ont., and Merrill Butler, of the Los Angeles Ry., Los Angeles, Calif. In Fig. 2, in the right-angle triangle OBX the radius OX and the tangent OB are known; hence BX and the angle OBX are determined. The angle YB2 is known and therefore the angle XB2. The solution of the triangle XB2 for B2 gives the required data for one corner. It should be noted, however, that two solutions are possible when two sides and an angle opposite one of them are known; the correct solution can, of course, be determined by inspection.

Fig. 3 gives another simple trigonometrical solution sent by J. Calvin Locke, of Brooklyn, N. Y. Through the center of the arc a line parallel to the tangent is drawn and the lot lines aa_1, bb_1, etc., extended to meet it. In the triangle o(P.T.)a, R and (P.T.)a are known. The tangent function gives the angle (P.T.)Oa, and the side Oa and the angle O(P.T.)a can be found. The angles

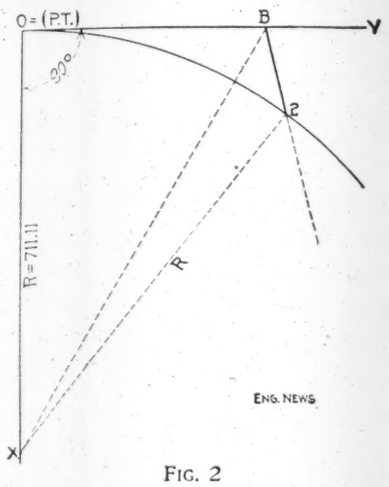


FIG. 2

at a', b', c', etc., are equal to corresponding ones at a, b, c, etc., which are known; therefore the triangles Oaa' and O1a' can be solved for aa' and 1a', which gives the required data for one corner.

For those who wish to use coördinates, E. P. Murray, of the Topographical Bureau of the Borough of Queens, New York City, recommends a rectangular system where the P.T. is selected as the origin and the radius through it is the X axis. The coördinates of a, b, c, etc., and of 1, 2, 3, etc., are easily computed from right-angle triangles. The directions of the lines connecting these points and the center of the circle are determined from their respective coördinates. The solution of two right-angle triangles then gives the required data for a single corner.

Another method of rectangular coordinates is given by H. C. Stevens, of Charlotte, N. C., in which the X axis

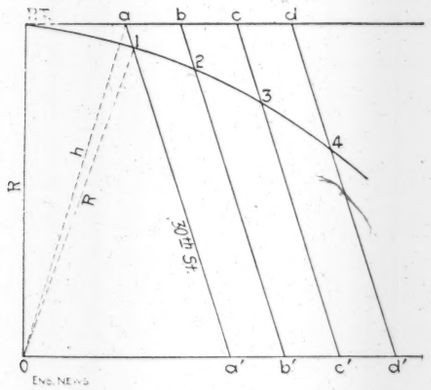


FIG. 3

is taken parallel to the lot lines aa_1, bb_1, etc., and the equation for the circle, whose center is at the origin is used, $x^2 + y^2 = R^2$. The coördinates of the P.T. are first computed, knowing the angle that the radius through P.T. makes with X axis, which we will call α. The general expression for the ordinates is $y = y_0 + m \cos \alpha$, where $y_0 = R \sin \alpha$, the ordinate of the P.T., and m is the tangent distance to the old lot corners a, b, c, etc. The abscissas of the new corners on the curve, 1, 2, 3, etc., are found from the equation

$$x = \sqrt{R^2 - y^2}$$

of the circle, and the abscissas of the points on the tangent, a, b, c, etc., are found from the equation

$$x' = y \tan \alpha - \frac{R}{\cos \alpha}$$

the equation for the tangent line. The difference of the abscissas for the same value of y, $x' - x$, are the lengths desired. This solution is interesting as being a demonstration of the practical use of analytical geometry.

Old Wooden Bridges in Ohio—A bridge census for Greene County, Ohio, was made by S. M. McKay, county surveyor, a short time ago. In reporting the figures he stated that 34 covered wooden bridges exist in the county, and many of these were built over 50 years ago. Most of the 34 are in fair shape, and if cared for will last years longer. Some (?) of them were not designed to carry the heavy loads to which they are now subjected, chiefly in the way of heavy traction engines. The bridges have Burr or Howe trusses. Usually the trusses are amply strong, but the floor system is weak.

Curved Pipe with Internal Pressure—An author of a paper before the American Institute of Mining Engineers (April, 1912, "Bulletin") makes an error which is not uncommon, and is worth notice. He suggests that for certain experiments (on the behavior of water and rock at high pressures and high temperatures) an apparatus be constructed as follows:

The water and other necessary ingredients could be placed in a tube bored out of high-speed steel. This could be lined with platinum and sealed by calking the open end with a platinum plug held in place by a steel screw-cap. The pressure [produced in the interior by heating the tube] could be measured by having the tube bent into a circle so that the two ends nearly meet; the distance between the two ends is then an index of the pressure within.

The idea involved is an analogy to the Bourdon gage, which, however, has a flattened tube, with smallest dimension in the plane of the curve. A bent tube of circular cross-section has no tendency to straighten, and the above suggested arrangement for measuring pressures is therefore bound to fail, except in case the circular bore becomes flattened in bending.

Similarly any elbow or other bend in a pipe will not tend to straighten when subjected to internal pressure. If the elbow is capped, the pressure on the cap exactly balances the resultant force which the internal pressure exerts on the elbow; if it is not capped but connects to a straight run of pipe, there is a tension between elbow and straight pipe equal to the pressure which would be exerted on a cap.

A bent tube whose cross-section is not circular will tend to become circular as internal pressure is applied. This lateral distortion changes the curvature of the bend; if the section tends to expand in the plane of the bend, the bend straightens, while if it expands transverse to this plane the curvature of the bend becomes sharper.

Table for Dimensions and Areas of Circular Segments

The accompanying table is sent to us by W. W. Frazier, of the engineering department of the La Belle Iron Works, Steubenville, Ohio, who explains its use as follows: Divide the chord of the segment by the radius of the segment, and find the nearest corresponding figures in the first column. Multiply the corresponding figures in the second column by the square of the radius to obtain the area of the segment. Multiply the figures in the third column by the radius to find the height of the segment. For dimensions given in inches the result will, of course, be in inches and square inches, and similarly with feet.

TABLE FOR DETERMINING THE AREAS OF CIRCULAR SEGMENTS

C = chord length. R = radius. H = height. Δ = central angle.

$\frac{C}{R}$	$\frac{A}{R^2}$	$\frac{H}{R}$	Δ	$\frac{C}{R}$	$\frac{A}{R^2}$	$\frac{H}{R}$	Δ
0.0175	0.00000	0.00004	1°	0.7815	0.04176	0.0795	46°
0.0349	0.00000	0.00015	2°	0.7975	0.04448	0.0829	47°
0.0524	0.00001	0.00034	3°	0.8135	0.04731	0.0865	48°
0.0698	0.00003	0.00061	4°	0.8294	0.05025	0.0900	49°
0.0872	0.00006	0.00095	5°	0.8452	0.05331	0.0937	50°
0.1047	0.00010	0.00137	6°	0.8610	0.05649	0.0974	51°
0.1221	0.00015	0.00187	7°	0.8767	0.05978	0.1012	52°
0.1395	0.00023	0.00244	8°	0.8924	0.06319	0.1051	53°
0.1569	0.00032	0.00308	9°	0.9080	0.06673	0.1090	54°
0.1743	0.00044	0.00381	10°	0.9235	0.07039	0.1130	55°
0.1917	0.00059	0.00460	11°	0.9389	0.07417	0.1171	56°
0.2091	0.00076	0.00548	12°	0.9543	0.07808	0.1212	57°
0.2264	0.00097	0.00643	13°	0.9696	0.08212	0.1254	58°
0.2437	0.00121	0.00745	14°	0.9848	0.08629	0.1296	59°
0.2611	0.00149	0.00856	15°	1.0000	0.09059	0.1340	60°
0.2783	0.00181	0.00973	16°	1.0151	0.09502	0.1384	61°
0.2956	0.00217	0.01098	17°	1.0301	0.09958	0.1428	62°
0.3129	0.00257	0.01231	18°	1.0450	0.10428	0.1474	63°
0.3301	0.00302	0.01371	19°	1.0598	0.10911	0.1520	64°
0.3473	0.00352	0.01519	20°	1.0746	0.11408	0.1566	65°
0.5345	0.01301	0.03637	31°	1.0893	0.11919	0.1613	66°
0.3816	0.00468	0.01837	22°	1.1039	0.12443	0.1661	67°
0.3987	0.00535	0.02008	23°	1.1184	0.12982	0.1710	68°
0.4158	0.00607	0.02185	24°	1.1328	0.13535	0.1759	69°
0.4329	0.00686	0.02370	25°	1.1472	0.14102	0.1808	70°
0.4499	0.00771	0.02563	26°	1.1614	0.14683	0.1859	71°
0.4669	0.00862	0.02763	27°	1.1756	0.15279	0.1910	72°
0.4838	0.00961	0.02969	28°	1.1896	0.15889	0.1961	73°
0.5008	0.01067	0.03185	29°	1.2036	0.16514	0.2014	74°
0.5176	0.01180	0.03407	30°	1.2175	0.17154	0.2066	75°
0.5345	0.01301	0.03637	31°	1.2313	0.17808	0.2120	76°
0.5512	0.01429	0.03874	32°	1.2450	0.18477	0.2174	77°
0.5680	0.01566	0.04118	33°	1.2586	0.19160	0.2229	78°
0.5847	0.01711	0.04370	34°	1.2722	0.19859	0.2284	79°
0.6014	0.01864	0.04628	35°	1.2856	0.20573	0.2340	80°
0.6180	0.02027	0.04894	36°	1.2989	0.21201	0.2396	81°
0.6346	0.02198	0.05168	37°	1.3121	0.22045	0.2453	82°
0.6511	0.02378	0.05448	38°	1.3252	0.22804	0.2510	83°
0.6676	0.02568	0.05736	39°	1.3383	0.23578	0.2569	84°
0.6840	0.02767	0.06031	40°	1.3512	0.24367	0.2627	85°
0.7004	0.02976	0.06333	41°	1.3640	0.25171	0.2686	86°
0.7167	0.03195	0.06642	42°	1.3767	0.25990	0.2746	87°
0.7330	0.03425	0.06958	43°	1.3893	0.26825	0.2807	88°
0.7492	0.03654	0.07281	44°	1.4018	0.27675	0.2867	89°
0.7654	0.03915	0.07612	45°	1.4142	0.28540	0.2929	90°

The Presidents of Four Great National Engineering Societies

For the 19th consecutive year, we present in this issue the portraits of the presidents of four great American engineering societies, accompanied by biographical sketches, each of which has been prepared by a well known engineer who is a personal friend of the man whose career he portrays and is familiar with his professional achievements.

John Augustus Ockerson, President of the American Society of Civil Engineers

By Branch H. Colby*

The recent election of John Augustus Ockerson as President of the American Society of Civil Engineers marks another advance in an interesting career which has been distinguished by varied responsibilities and recognized by numerous honors.

A man elected to the highest office in any one of our four great national engineering societies must be a man who has achieved things, and who has been able to focus the attention of the mass of his fellow workers in his profession upon these achievements.

Weak men sometimes gain prominence and attain influence and position through powerful business or political organizations which seek a puppet subservient to their will; often a shrewd man by sharp practice or intrigue may for a time gain wealth, power and influence; but only men of sterling worth and integrity receive honors for a great work faithfully performed.

Especially is this true in our profession, overflowing with talented and enthusiastic workers, where good work is so common as to cause no comment, where excellent work is always expected, and only very difficult achievement or notably successful departure from old lines of practice is applauded.

We recognize and estimate the real worth of an intimate friend long before it can come to the knowledge of the multitude. An intimate acquaintance of 30 years, dotted at intervals by periods of professional association and collaboration, has given the writer rare opportunities to know Mr. Ockerson as a man and as an engineer. As a man, he possesses all of those qualities of mind, manner and speech which endear one man to another. Generous praise is spontaneous in eulogy of such a friend, and words are often inadequate to convey feeling or express appreciation.

Mr. Ockerson has received the highest honor in the power of the American Society of Civil Engineers to bestow. Those who have been favored by his personal acquaintance and are familiar with his professional record know that the honor was worthily conferred and will he humbly worn by one who is known to be one of the most modest of engineers. Almost retiring in manner, Mr. Ockerson never pushes himself forward for preferment, never seeks disciples to propagate his ideas or promote his plans. A student of methods, he seeks the way and means to accomplish his purposes. Clear in conception, quick in execution, he sees realized the creations of his master mind.

So varied have been Mr. Ockerson's activities that details cannot be given, but the "hard-boiled" facts of his career are interesting and significant. As his name would indicate, he is of Swedish origin. He was born in Skane, Sweden, on Mar. 4, 1848, and was the son of Jons Akerson and Boel Jons Dotler. Both parents died in 1851, within a few days of each other.

In the same year their son was brought to the United States, at so early an age that he has been to all intents and purposes an American all his life, further emphasizing his feeling for his adopted country by giving the Swedish name Akerson its more American form, Ockerson, and he served his adopted country in the 132d Illinois Infantry and the First Minnesota Artillery during the last two years of our Civil War.

He was graduated from Elmwood (Illinois) High School, and matriculated at the University of Illinois, receiving from that institution the degree of Bachelor of Science in Civil Engineering, in due time that of Civil Engineer, and in the year 1903 the honorary degree of Doctor of Engineering. While at the University he took an active part in student activities, as a member of Tau Beta Pi, President of the Adelphi Society, and President of College Government.

Leaving college, his first engineering experience was as transitman with the Atchison, Topeka & Santa Fé Ry., after which he became Assistant Engineer with the U. S. Lake Survey. In 1867 we find him Assistant Engineer upon the Eads Jetties, followed by several years' service as Assistant Engineer under the Mississippi River Commission.

Leaving the Mississippi River Commission, he became Engineer and Manager of the Silver Age Mining Co., and after a successful experience in that capacity was appointed Principal Assistant Engineer by the Mississippi River Commission; and in 1898, by appointment of the President of the United States, he became a member of that Commission, a position to which he brought a ripe practical experience.

This position he has filled with signal ability for many years. To the work of this Commission he continues to give his best thought and effort. In 1910 he was placed in charge of the Colorado River work, and continues to give much of his time to that service.

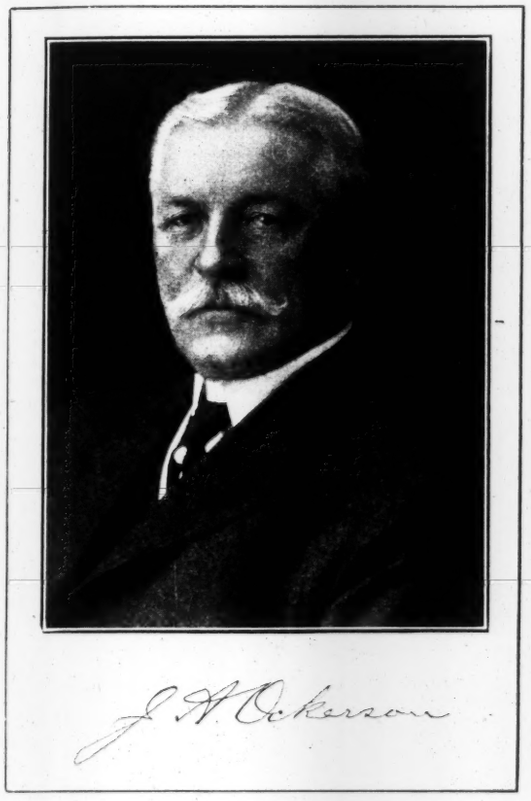
Long before this time, Mr. Ockerson's position had become a prominent one, as was evidenced by his appointment to the Jury of Awards of the Paris Exposition in 1900, and as Chief of the Department of Liberal Arts of the Louisiana Purchase Exposition from 1902 to 1905, and as member of the Superior Jury of Awards. In 1900 the United States sent him as a delegate to the International Congress of Merchant Marine in Paris; in 1903 he was Resident Commissioner for Sweden at the Louisiana Purchase Exposition; and in 1905, United States Delegate to the Tenth International Congress of Navigation, at Milano, Italy, and at St. Petersburg in 1908.

Decorations have been awarded Mr. Ockerson by the following countries: Germany, Officer of the Crown; Italy, Knight of the Crown; Sweden, Knight of the Order of Vasa, and Commander, Order of the Vasa; France, Officer Beaux Arts; Belgium, Knight of the Order of Leopold; China, Order of the Double Dragon, first grade.

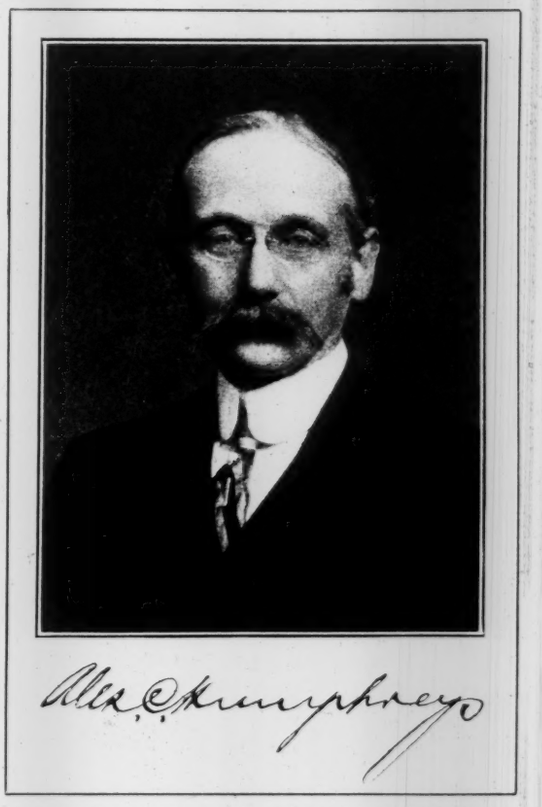
It might seem that a man active in so many branches would have little time to spare for anything else, but Mr. Ockerson is well known also as a member and officer of numerous societies and as a contributor to engineering publications. In 1898, he was elected a member of the National Geographical Society; in 1880, of the American Society of Civil Engineers, of which he has been Vice-President. He is a member of the St. Louis Engineers' Club, and has twice been its Vice-President and twice its President. Other organizations of which he is a member are: Ransom Post 131, Grand Army of the Republic; Bellerive Country Club of St. Louis; Noonday Club, St. Louis; and University of Illinois Alumni Association of St. Louis, of which he is President.

His publications include numerous papers contributed to the *Transactions* of the American Society of Civil Engineers; the *Journal* of the Associated Engineering Societies; the *Engineering Journal;*

*Security Building, St. Louis, Mo.



PRESIDENT OF THE AMERICAN SOCIETY
OF CIVIL ENGINEERS



PRESIDENT OF THE AMERICAN SOCIETY
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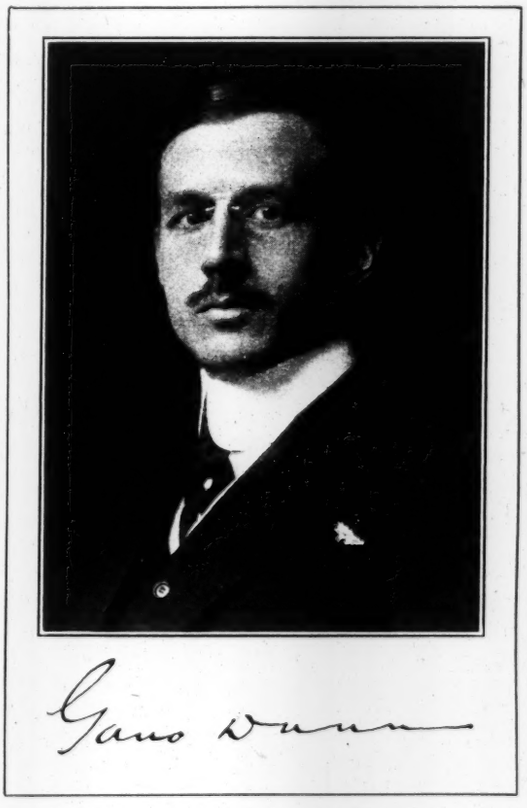
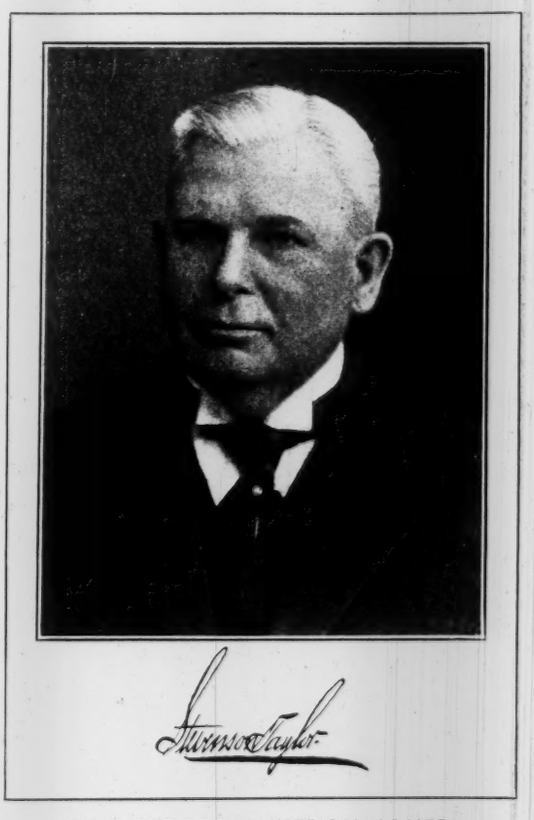


PRESIDENT OF THE AMERICAN INSTITUTE
OF ELECTRICAL ENGINEERS



PRESIDENT OF THE AMERICAN SOCIETY
OF NAVAL ENGINEERS AND MARINE ARCHITECTS

the *Proceedings* of the Eighth International Congress of Navigation, Milan, 1905, and the Eleventh Congress, St. Petersburg, 1908; Engineering Congress, Glasgow, Scotland, 1901, and official reports.

He was married, Nov. 3, 1875, to Miss Helen M. Chapin, of Detroit, Mich. She died in March, 1886. On June 4, 1890, he was married to Miss Clara W. Shackelford, of St. Louis, Mo., and now resides at 501 Clara Avenue, St. Louis.

Alexander Crombie Humphreys, President of the American Society of Mechanical Engineers

By Alten S. Miller*

The American Society of Mechanical Engineers is particularly fortunate in securing for its President the man who is doing as much for mechanical engineering as any other living person. As President of the only institution of learning in this country devoted exclusively to mechanical engineering, Dr. Humphreys is taking every advantage of his opportunities to aid the profession, and as a Trustee and member of the Executive Committee of the Carnegie Foundation for the Advancement of Teaching, his influence is felt in every higher institution of learning in the United States.

Alexander Crombie Humphreys was born in Edinburgh, Scotland, Mar. 30, 1851. His father, E. R. Humphreys, was an English classical scholar and a well known educator, and had the degrees of M. D., M. A. and LL. D. Mrs. E. R. Humphreys was a member of the McNutt family of Prince Edwards Island, Canada.

Dr. E. R. Humphreys moved with his family in 1859 to Boston where he took out letters of naturalization, thus naturalizing his minor children. Here Alexander Crombie Humphreys continued his schooling until he was 14, when he prepared to go into the United States Navy.

Having passed the preliminary examinations to the United States Naval Academy, Mr. Humphreys was barred by his youth and then decided to enter business until he was qualified by age to enter the Naval Academy. Accordingly he began work in an insurance office in Boston and in 1866 went to New York in the employ of the Guarantee & Indemnity Co. of that city. Having become established with that company he abandoned his plan of entering the Navy, and by 1872, at the age of 21, he had reached a post of high responsibility.

In that year he married Miss Eva Guillaudeu, daughter of the late Dr. Emile Guillaudeu, of New York. About this time he was made Secretary, and shortly afterward Superintendent of the Bayonne & Greenville Gas Light Co.

In 1877 Mr. Humphreys entered the Stevens Institute of Technology and graduated in 1881. During this period he continued his work of Secretary and Superintendent of the Bayonne & Greenville Gas Light Co. Shortly after his graduation Mr. Humphreys was appointed Chief Engineer of the Pintsch Lighting Co., of New York, and made a special study of the manufacture and utilization of gases of high illuminating power for the lighting of trains, boats, buoys, etc.

In 1885 Mr. Humphreys was appointed Superintendent of construction for the United Gas Improvement Co., of Philadelphia. Soon after entering its service he was made General Superintendent of the company, and in this position soon assumed control of the contracting and purchasing departments, and of the commercial management. During the period from 1885 to 1892 Mr. Humphreys had under his control numerous gas and electric-light properties, increasing from ten at the beginning to about forty in 1892. It was not only necessary in many cases to rehabilitate the physical properties but in most cases Mr. Humphreys completely reorganized the operating departments, and in every case not only increased the efficiency, but raised the standard of management and gained the good will of the public.

While rebuilding and reorganizing these gas and electric properties, Mr. Humphreys reorganized and reduced the management to a system. At the same time he took charge of a very large and profitable contracting business which the company had developed during the period of his incumbency. Not the least of his accomplishments during this period was the bringing of water-gas apparatus from a small beginning to a very high state of perfection.

In the late '80s he installed a storage battery to be operated in connection with a central-station lighting plant which was one of the earliest, if not the earliest battery used for this purpose.

In 1892 Mr. Humphreys joined with his friend and associate, Mr. A. G. Glasgow, to establish the firm of Humphreys & Glasgow, of London, and in 1894 he left the services of the United Gas Improvement Co. to take charge of the work of the firm of Humphreys & Glasgow, of New York.

The business of the firm was incorporated under the name of Humphreys & Glasgow, Inc., in 1909 and was changed to Humphreys & Miller, Inc., a few months ago.

In addition to taking care of the work in connection with his engineering firm. Dr. Humphreys has been very active, since he moved to New York, in many lines and especially so in connection with Stevens Institute, of which he was elected President in 1902. As in every other case where he has taken up special work, he has gone into it heart and soul, and the results accomplished fully justify the effort expended. The work of the Institute has been immensely broadened and the efficiency greatly increased. Large additions have been made to the property, buildings and equipment, and not only has the capacity of the Institute been increased and the standard of education raised, but much has been done for the comfort and health of the student body.

Dr. Humphreys has received honorary degrees of Sc. D. from the University of Pennsylvania, and LL. D. from Columbia University, New York University and Princeton University. Besides being President of Humphreys & Miller, Inc., he is President of the Faculty and of the Trustees of Stevens Institute, of the Buffalo Gas Co., the Engineers Club of New York and the Canadian Society of New York; and he is Past-President of the American Gas Light Association, of the American Gas Institute, and of the National Society for the Promotion of Industrial Education.

Dr. Humphreys is a member of the Council of the American Institute of Mining Engineers, and is a member of the American Society of Civil Engineers, of the Institution of Civil Engineers, of the British Association for the Advancement of Science, of the American Association for the Advancement of Science, of the National Society for Promoting Engineering Education, and of the Engineers, the Lotus, Union League, Century, Lawyers, Chemists and Church Clubs, of New York, of the University Club, of Philadelphia, and is associated with many other organizations.

Dr. Humphreys takes a deep interest in accounting, lecturing to his students on this subject, and laying great stress upon its importance to all engineers. His book upon "The Business Features of Engineering Practice" is widely known.

It is easy for one who has had even a brief association with Dr. Humphreys, to understand how he is able to accomplish so much more than the average man. He has an almost unlimited capacity for work, together with a magnificent physique, which enable him to undergo physical and mental strain that few men could endure. His force, high standards, and personal magnetism attract all who know him, and inspire others to work for him as they would not work for themselves. With all associated with him, working for him, and along the lines laid down by him, it is easy to see the immense amount that can be accomplished in a short time. Dr. Humphreys has the courage of his convictions, recognizes no obstacle, and never takes a step without fully understanding where it will lead. He has an infinite capacity for detail and never takes up any subject without going to the bottom of it. Dr.

*165 Broadway, New York City.

Humphreys is constructive, and is in no sense an iconoclast, and no man could be better qualified to assist in the good work that is being done by the American Society of Mechanical Engineers.

Gano Dunn, President of the American Institute of Electrical Engineers

By Frederick Remsen Hutton*

It is an interesting evidence of the intense energy characterizing the electrical engineering profession that a specialist in the problems of the generation and distribution of electrical energy like Mr. Dunn, who was born no longer ago than 1870, should have entered the profession by the door of the young telegraph operator. In this he is like Edison, like the elder and younger Pope, like Brush and Weston and many other electrical engineers of distinction.

In the era of Morse and Lefferts there were no other avenues open to the student of electrical engineering. Before the organization of the American Institute of Electrical Engineers, before the day of the central station and the telephone, this was the only way in.

Mr. Dunn used his skill as an operator in night service in self support while a student for the B. S. degree at the College of the City of New York, in 1889, and while pursuing his specialized electrical engineering study for the E. E. degree at Columbia University in 1891.

The course at Columbia was then two years long only, but was restricted to graduates from previous engineering courses. It was not until some years later that it was opened to graduates of the high school and made four years long. Mr. Dunn came under the strong stimulus of the work of those early developing years, with Francis B. Crocker and Michael Pupin and their interested co-workers.

The pleasant relations between Prof. Crocker and President Schuyler Skaats Wheeler of the rising industry at Ampere, N. J., resulted in the opportunity for the young graduate to win his spurs in experimental work with the Crocker-Wheeler Co., and he rose from subordinate to responsible positions, from assistant engineer to chief engineer, vice-president and director. This scope of duty made him responsible designer both of direct- and alternating-current work, motors, generators, transformers and other apparatus and machinery. The problem of generating, transmitting, transforming and applying power in electrical form has thus felt his formative hand in all of the productive industries.

His first alma mater (C. C. of N. Y.) gave him the honorary degree of Master of Science in 1897, both for his practical achievements and for a master's dissertation, "The Distribution of Electric Power in Factories."

But Mr. Dunn is not the self-centered designer of the cloistered and isolated drawing room. He is a representative of the highest type of the man of affairs, who can enter an arena of competition in salesmanship, and by his complete knowledge, his genial address, his way with men, he can convince the investor that the proposed installation can safely be made, and its success assured.

It was these qualities which secured for his company some most important opportunities, such for example as the gas-engine driven plant of 12,000-kw. alternators for the California Gas and Electric Corporation and other similar important contracts.

The active development of power service during Mr. Dunn's professional life has given him abundant opportunity to exercise his inventive ability. His successes include inventions in the structural features of the dynamo, motor, rheostat and heavy-power switch, and in systems of operation, control and speed regulation. An example of this sort of achievement is a system for operating heavy-duty two-high reversing roll trains in steel mills, so that a full speed forward can be changed to full speed in reverse direction within one second without excessive demand peaks being noticeable at the operating station. The time of reversal is shorter than ever before attained and there is also such a subdivision of operating parts that the failure of one such detail does not shut down the mill. Patents have been recently allowed to Mr. Dunn for a system and machinery for winding thin copper ribbon on edge; he has secured far higher speed, the bending around shorter radii than before, and he has eliminated the tendency of the thin metal coil to go out of form by springing back after the coil is made.

His professional colleagues have recognized his ability both as an engineer, a man of judgment in affairs and a helpful co-worker by loading him with the honors of office. He has been a manager of the Institute of Electrical Engineers in 1897 and 1902; a vice-president in 1900 and 1905, and was elected president in 1911. He has served the Institute most acceptably on important committees, such as its Constitutional Revision, its High-Tension Transmission, its editing and building committees, and is now representing the Institute on the Board of Trustees of the United Engineering Society, of which he is a vice-president. He has been one of the workers on the International Electro-technical Commission, and the Power Transmission Committee of the National Electric Light Association.

On the social side, he belongs to athletic organizations—he is an enthusiastic fencer—is a member of the University and the Engineers' Club, and of the select and distinguished professional organizations germane to his line of work.

In November, 1911, Mr. Dunn was elected a director and vice-president in charge of engineering and construction of J. G. White & Co., operating directly and through associated companies in the United States and Canada, Europe, South America and the Philippines, concerned with hydro-electric developments, central lighting and power stations, electric and steam railways and all kinds of public utilities and service.

It has been a great pleasure to one who, like the writer, has watched Mr. Dunn's steady growth in power and responsibility and capacity for the bearing of burdens to prepare this notice, and to feel that at one time it was his privilege to have a share in a preparation for so effective a life work.

*Late Professor of Mechanical Engineering, Columbia University; Secretary Amer. Soc'y Mechanical Engineers, 1883-1906; 257 West 86th St., New York City.

Stevenson Taylor, President of the Society of Naval Architects and Marine Engineers

By Walter M. McFarland*

It is always assumed that the president of a society is one of its most eminent members, and never has this been truer than in the case of Mr. Stevenson Taylor, President of the Society of Naval Architects and Marine Engineers. When the nomination of Mr. Taylor for the Presidency was made by the Council in 1909, no other name was considered, and his election by the Society was unanimous amid great applause. He acted as President, during the absence of the former incumbent (Admiral Bowles) at the 1909 meeting, so that with the expiration of his term at the end of 1912, he will have been practically president for four years.

Mr. Taylor has long been one of the most distinguished marine engineers in the United States, and in his particular specialty, the design and construction of paddle-wheel machinery, he is undoubtedly the greatest expert in the world. When it is mentioned that he has had a very responsible part in the design of such vessels as the "Albany"; the "New York," of the Hudson River Day Line; the "City of Detroit" and others of the Detroit-Cleveland Line; the "City of Buffalo" and "City of Erie," of the Cleveland-Buffalo Line; the "City of Fall River," the "Puritan," and others of the Fall River Line; the "City of Bangor" and "City of Rockland," of the Boston-Bangor Line, and the "Massachu-

*The Babcock & Wilcox Co., 85 Liberty St., New York City.

setts," "Bunker Hill" and "Old Colony" of the Boston-New York Line, even the layman recognizes his preëminence.

Mr. Taylor is one of those rather rare people at the present day, a real New Yorker by birth as well as location. He was born in Hammersley, now West Houston St., in 1848, and moved to Greenwich St., near Desbrosses, early in the '50's. His education was entirely in the public schools of New York, including a period at the Free Academy, now called the College of the City of New York. In his sixteenth year, having decided to follow a business rather than a professional career, he left the City College and having studied stenography, secured a position with the firm of Dexter, Hamilton, Johnson & Farrelly, which later developed into the American News Co. His father had the old notion, which is still prevalent to some extent, that every boy should learn a trade, whatever else he might do, so as to "have something to fall back on." A schoolmate of Mr. Taylor's was a Harrison, the brother of a member of the firm of Fletcher, Harrison & Co., proprietors of the North River Iron Works, whose establishment was on West St., near Desbrosses, and only two blocks away from where he lived. Under the circumstances it was very natural that he should turn to this establishment as a place to learn a trade, and in May, 1864, he became an apprentice to the firm of Fletcher, Harrison & Co., to learn the trade of machinist or engine builder.

His first work was in the drawing room, under the direction of the late Andrew Fletcher, varied after the first year with periods of shop work and erection work, but he had shown such aptitude as a designer and draftsman that before the end of his apprenticeship he became permanently attached to the drafting room. At first he was the assistant to Mr. Fletcher, who gave his personal attention to this work, but after a time Mr. Taylor became the responsible head of the drafting department. During about two years of his apprenticeship he was a teacher in a public high school.

It is interesting to note that the North River Iron Works have an enviable record as having constructed the machinery for some of the most famous river and Sound steamers, such as the "Mary Powell," '61-2; "Chauncey Vibbard," 1864; the "Nautilus," afterward the "Stonington," 1866; and others that were mentioned above. The firm also had an extended experience in building the machinery for lake steamers, and Mr. Taylor's contact with this work began with the "R. N. Rice," in 1867, going down to the "City of Erie," in 1898, and including many famous ones, such as the "City of Detroit," "City of Chicago" and "City of Buffalo," a number of which had compound engines. To those who know him, it is not at all surprising that his employers very early learned to repose the greatest confidence in him, and as a result he was given an opportunity of becoming familiar with every branch of the business; that is, the construction and erection of the machinery, and the financial side as well as the design, which was his own special field.

It is only natural that a young man of his marked ability and charming personality should have attracted the attention of others besides his employers, with the result that several opportunities were offered him to engage in business with other concerns, and one to go to Turkey, in 1876, to superintend the building of some steamers there. However, his affection for his old concern was too great to let him go elsewhere.

In 1878 the North River Iron Works made a contract with Peirce, Montgomery & Howard, shipbuilders, of Chelsea, Mass., for beam engines for the "Nahant" and "Nantasket." This work and contracts for other machinery which followed brought the firm and Mr. Taylor in very close touch with Mr. George Peirce, who later became Supervisor of Marine Construction for the Old Colony Steamboat Co. In 1881 Mr. Peirce was instructed to procure estimates for the new Fall River Steamer, "Pilgrim," which was to be much larger, faster, and more commodious than the "Bristol and "Providence," then the "Queens of the Sound," and he requested Mr. Andrew Fletcher and Mr. Taylor to assist him in preparing the plans and specifications. The contract was awarded to the Morgan Iron Works, of New York. The "Pilgrim" was one of the first steamers in the world to use electric lighting and the installation was made under Mr. Taylor's supervision, at the request of Mr. Peirce. It consisted of 869 lamps of about 11,000 candlepower, which was the largest equipment by far installed on any vessel or in any place on shore up to that date.

In 1883 the former partnership (which had become W. & A. Fletcher by the withdrawal of Mr. Harrison in 1881) was changed to a corporation consequent upon the death of Mr. William Fletcher, and Mr. Taylor was made vice-president of the new corporation, continuing as the responsible technical head thereof, Mr. Andrew Fletcher choosing to reduce his activities.

The famous Sound steamer, "Puritan," was the first of the Fall River Line for which the North River Iron Works took the entire contract, and for the machinery of this vessel Mr. Taylor made the designs in 1886. The hull was built by Roach's Yard, at Chester, Penn., and the larger parts of the machinery at the Morgan Iron Works, the assembling all being done by the Fletchers.

In 1904 he disposed of his interest in the W. & A. Fletcher Co., and was then appointed by the Receiver of the United States Shipbuilding Co. to appraise the several yards and other property of that concern. This service was completed in July. In August, 1904, at the request of his life-long friend, Mr. N. F. Palmer, and the latter's sons, he joined the Quintard Iron Works Co., as vice-president, which position he still holds.

If it were proper to call a moving object a monument, it might be said that the greatest monument at present to Mr. Taylor's ability is the steamer "Commonwealth," of the Fall River Line, which is universally regarded as the highest type of luxurious passenger steamer in the world. Such thorough satisfaction had been given by all the work which he had done on previous vessels of the Line that the contract with the Quintard Iron Works Co., for the "Commonwealth" was concluded after a conference of about an hour in the fall of 1906 between him and the president and the general manager of the company. This contract included everything about the vessel, even to the tableware. The only conditions laid down by the Navigation company were as to width, beam and draft, which were determined by the conditions of the service. A paper describing the "Commonwealth" was read before the Society in 1908, and in the discussion a prominent engineer who is himself an authority on vessels of this character, said:

> The development of this type of vessel has been largely due to a master mind, a man who from boyhood has made the development of the American steamboat his special study and has brought it to what appears to be perfection. I refer to one of our vice-presidents, Mr. Stevenson Taylor, to whom we all feel under great obligation for producing this masterpiece of steamboat construction.

Capt. J. W. Miller, who was then general manager of the Fall River Line, also paid the highest compliments to Mr. Taylor's ability in the design and construction of the "Commonwealth."

Besides his strictly professional work, Mr. Taylor has had a very active part in a great philanthrophic work connected with shipbuilding. He was consulted by Mr. William H. Webb, the famous shipbuilder of the '50's and '60's, with reference to the establishment and endowment of Webb's Academy and Home for Shipbuilders, and upon the establishment of that institution was made a charter member of the Board of Trustees. Of the eleven member of this original board, only Mr. Taylor and Mr. Charles H. Cramp survive.

After the death in October, 1899, of Mr. Webb, who was President of the Board, Mr. Taylor, who had been Vice-President for several years, was made President, and he has been annually re-elected to that office since. For the benefit of those who do not know the scope of this charity it may be mentioned that it combines a home for aged and indigent workmen connected with ship and marine-engine builders, and their wives and widows, with an academy for training young men in the theory and prac-

tice of shipbuilding and marine-engine building.

The young men have to pass an entrance examination, and during their attendance at the academy are provided free not only with instruction, but with board and lodging as well. Since its foundation 128 students have been graduated, and there are now 40 in the Academy. There are at the present time 47 guests in the Home. Beside the real estate and buildings at Fordham Heights, the institution has real estate and other property valued at more than a million and a half of dollars, with an annual income of about $70,000. It may be mentioned that the officers and members of the Board of Trustees receive no remuneration.

Mr. Taylor was one of the first members of the Society of Naval Architects and Marine Engineers, and has been a member of its Council from the very beginning. For some years prior to his election to the Presidency he was one of the Vice-Presidents. He has always been very active in its affairs, never missing a meeting, and contributing very materially to the value of the Transaction by his part in the proceedings. He is a born harmonizer, and on several occasions, when discussions have tended to take an acrimonius turn, his tact and good humor have brought affairs back to a normal condition.

He was one of the delegates representing the Society at the International Congress of Naval Architects and Marine Engineers in July of last year, at London, under the auspices of the Institution of Naval Architects, where he most worthily represented the Society and the United States as one of its foremost engineers.

It has been the writer's pleasant task to contribute a biographical sketch such as this for several men of great eminence professionally, and it has always been a pleasure to him to be able to praise the subject of the sketch on the human as well as on the professional side. Indeed, the writer would find it almost impossible to attempt to praise a man professionally whom he could not respect and admire personally. He has had the privilege for many years of counting Mr. Taylor as one of his dearest friends, and it is very difficult for him to speak of his lovable qualities without appearing to exaggerate. In all his acquaintance, he has never known a man who more thoroughly enjoyed and deserved the real affection of his associates and those who know him best than Mr. Taylor. He is a man of distinguished appearance, gracing any occasion, and the Society has counted itself very fortunate in having him as its presiding officer at its recent banquets. He has a delightful sense of humor and is a skilled raconteur worthy to rank with the most famous. An evidence of his great personal popularity is the fact that he has been for many years one of the Board of Governors of the Engineers' Culb, of which he is now vice-president.

The rules of the Society limit the term of president to three years and make him ineligible for immediate reëlection. If it were not for this, it is safe to say that Mr. Taylor would be elected to hold the office as long as he is willing to fill it.

Letters to the Editor

The Government Investigation of Scientific Management

Sir—It is with great interest that I have read your editorial in the issue of Mar. 28, 1912, upon the governmental investigation of scientific management and also the report of the investigating committee.

From observations which I have made during the past ten years, I believe that the scientific management of any corporation, company, shop or small body of men depends entirely upon the perceptive and logical reasoning powers of those in charge. As regards the thoroughness and dispatch with which any particular piece of work may be done, no one is in a better position to determine the best manner of accomplishing the results than the workman who is constantly engaged upon the particular work. Until recently, however, workmen have had little or no incentive to advance their ideas for more efficient means of improving or shortening the routine methods. In fact I have heard of only one company that has realized the possible value of the ideas of their employees sufficiently to solicit these ideas and extend compensation.

Generally speaking, workmen are assigned to perform their duties in certain ways and have no alternative but to do as directed. As a result of these "not-to-reason-why-but-do-or-die" methods, duties are performed mechanically, and anyone who works mechanically will never attain the height of efficiency. It is obvious, of course, that it would be impossible for every workman, when assigned a certain duty, to enter upon an argument or discussion as to the best ways and means of performing this duty. A proper time and place should be set aside in which all workmen, and also those in charge, could be given an opportunity to submit their ideas, opinions or suggestions before a committee of those who are financially interested.

I have suggested a committee, to receive and investigate these ideas, as I do not believe any one man is completely capable of passing judgment upon the merits of any new idea. Then again, there is nothing more discouraging and disgusting to a workman, who has had the courage to advance his ideas to his immediate superior, than to have these ideas quietly ignored, mainly, perhaps, because the majority of these persons in immediate charge are loth to admit the possibility of their methods being improved upon, as it might reflect upon their ability of being completely capable and "up-to-the-minute."

The average workman is only too glad to advance his ideas when he is sure that they will be appreciated and credited to him by the proper persons. On the other hand, a workman having any doubts as to the reception of his ideas will undoubtedly keep mum and as close as an oyster, and perform his duties in a mechanically plodding way. It is thus that a great many worthy ideas are lost.

Standardizing and systematizing, for any particular shop, can best be done by those who are continually at work upon the duties to be so arranged. Stimulation, in any form, can hardly be a complete success. A workman who performs his duties under "Simon Legree" methods or through the fear of being discharged will, as I have before stated, plod along mechanically. A "speeder" or "pacemaker" will breed ill-feeling and discontent among any body of workmen. A workman who conscientiously tries to increase his speed above its normal rate, such as those who are paid by piecework or receive bonuses, will greatly increase his percentage of personal error and also eventually become a physical or mental wreck, with a tendency to shorten his life.

A man is most efficient, I believe, when allowed to work at his own normal rate of speed and by his own particular methods, assuming that the results are satisfactory. No body of men can be found, I am sure, who can work as a whole at the same rate of speed and still have each individual accomplish results of the same quality in a given time. There may be one or two, or perhaps a few, that will be

capable of this. Others will be found who are slower or perhaps less thorough and exact. The only way of dealing with these differences in abilities is by differencies in compensation.

Any company or individual that begins a "greater efficiency" movement by a "weeding out" of employees will always find themselves short-handed. Every shop must take its good, bad and indifferent classes of workmen, just as we must all take the good, bad and middling when we buy a peck of apples. Every shop cannot have first-class, A-1 men entirely. All men are not up to a certain standard, yet every man is entitled to, and must have, a chance to gain a livelihood.

T. N. ASHTON.

48 Rathburn Ave.,
White Plains, N. Y., Apr. 4, 1912.

The Charges against Mr. Charles G. Elliott

Sir—It is gratifying to note your espousal of the cause of that honest, conscientious man, Charles G. Elliott, now being persecuted because he would not become a party to fraud, nor the tool of the unscrupulous boomers of a project that is without merit.

I inclose copy of resolutions which I presented to the Second Annual Session of the National Drainage Congress in New Orleans on Apr. 12. These resolutions were received with applause; but a gentleman from Georgia opposed them upon the grounds that they were ill-timed, as a Committee of Congress was investigating the charges against Mr. Elliott and therefore no effort should be made to influence that investigation. This gave rise to considerable discussion, which was terminated by John Fox, who cited a provision of the Constitution of the Congress which barred this class of resolutions. Mr. Fox paid a well deserved tribute to Mr. Elliott as a man and an engineer.

Now, the question arises, what ought the American Society of Civil Engineers to do when the power of one bureau of the government is being used to crush one of its members, who has been in good standing for 32 years, and has sustained a reputation which is above reproach?

ISHAM RANDOLPH.

826 First National Bank Bldg., Chicago,
Apr. 19, 1912.

RESOLUTIONS PRESENTED TO THE SECOND NATIONAL DRAINAGE CONGRESS

Whereas, Charles G. Elliott, Chief of the Bureau of Drainage Investigations of the Department of Agriculture, and A. D. Morehouse, Assistant Chief of the same bureau, have been dismissed from the United States service on the charge that they had certified and presented false accounts on which the government paid out money; and whereas, the alleged offense consisted solely of borrowing funds from the 1910 appropriation to pay bills incurred under proper authority in 1909, said borrowing being made necessary because Mr. Elliott's superiors failed to reserve for the use of his work, funds which he had been authorized by Congress and by his superiors to spend; therefore be it resolved, by the Second National Drainage Congress, that Mr. Elliott and Mr. Morehouse are unjustly discharged; that they have only been guilty of technical misdemeanors which are in common usage in every department of our government; and be it furthur resolved, that we hereby testify our belief in the personal integrity of Mr. Elliott and Mr. Morehouse, our knowledge of their ability as drainage engineers, our high regard for their brave defense of innocent investors; and be it further resolved, that we believe a fair and full investigation of their offices and acts will wholly absolve and exculpate them and reinstate them to the good of the service; and be it further resolved, that these resolutions be spread upon our records and copies of them be sent to the President, to the members of the Senate and House of Representativs of the United States.

New Orleans, Apr. 13, 1912.

Wood-Stave Pipe and Concrete Pipe

Sir—In the article of E. A. Moritz appearing in the issue of Mar. 28, 1912, entitled, "Description of the Prosser Division of the Sunnyside Unit, Yakima Project, U. S. R. S.," the author expresses surprise at the amount of shrinkage in the fir staves of the wooden pipe line. The cause probably is that the seasoning in the dry kilns is not entirely effective, and the extra shrinkage at the ends of staves would be expected as the sap has freer exit where the grain is cut across than in the body of the stave. That this shrinkage took place before the trench was backfilled is fortunate, in that the staves have received the additional seasoning. This shrinkage is commonly observed in but partially seasoned lumber in buildings, and unseasoned fir has the reputation of being a short-lived wood when covered with dry earth.

The unusual thickness of the staves is sure to be detrimental to the life of the pipe, as the head at both ends of the pipe is too light to cause a penetration of over ½ in. from the inside. The sapwood staves, if any have been inadvertently placed in the pipe, will have the longest life; this has been proven in many Douglas Fir pipe lines that have failed from dry rot. The clear flatgrain fir harbors the sap between the pitch rings and dry rot attacks it if the pipe is covered with dry filling, while the sapwood keeps saturated from the static pressure within the pipe.

The determination of the relative sizes of the concrete and wood-stave sections of this pipe line is perhaps open to discussion. The size selected for the concrete pipe was 30½ in., and for the wood pipe it was 31 in. Regardless of any refinements used in the calculations, this result is not in harmony with the accepted fact that the friction coefficient is greater for concrete pipe laid in lengths than in well laid continuous stave pipe, especially where curves in the alignment throw the joints apart on the outside of the curve. This difference will become even more marked after the pipe has been in use for some time, as the wood pipe will not lose in carrying capacity, while the concrete may be roughened by sand in the water, or a growth of algæ may impede the flow. I assume that an attempt was made to secure sections that would offer equal resistance to the flow in each case; but this may not be the case, as the line is not laid on the hydraulic grade line, and other reasons may have determined the relative size of the two kinds of pipe.

WINFIELD HOLBROOK.

Penrose, Colo., Apr. 6, 1912.

Some Contradictory Retaining Wall Results

Sir—In the hope that it will provoke discussion productive of additional information on the subject of retaining-wall pressures the following is presented:

1. In connection with certain grade-elimination work, retaining walls had to be designed by the writer for various conditions of pressure. One was that of a height of fill 20 ft. above footing at wall and sloping 1½ horizontal to 1 vertical to a considerable height beyond elevation of top of wall.

Using the accepted formulas for pressure for this case ($\theta = \phi$),

$$P = \frac{wh^2}{2} \cos \phi$$

$$H = P \cos \phi = \frac{wh^2}{2} \cos^2 \phi$$

for a wall of 20 ft. height and a slope of 1½ to 1,

$$H = \frac{100 \times 20^2 \times 0.832^2}{2} = 13,865 \text{ lb.}$$

If this same wall were to be designed to retain a fill up to the level of the top of the wall and from that point a uniform surcharge equal to the height A-B in figure, which is

$$\frac{20 \times \tan \frac{1}{2}(90 - \phi)}{\cot \phi - \tan \frac{1}{2}(90 - \phi)} = \frac{20 \times 0.535}{1.5 - 0.535} = 11.09$$

we obtain by the use of the following formulas deduced by Mr. Serber (ENGINEERING NEWS, Aug. 23, 1906) and verified by Mr. Cain ("Retaining Walls," 1909.)

$$P \text{ (for surcharge)} = wh'h \tan^2 \tfrac{1}{2}(90 - \phi) = H$$

$$P \text{ (for fill)} = \frac{wh^2}{2} \tan^2 \tfrac{1}{2}(90 - \phi) = H$$

$$\text{Total } H = wh \tan^2 \tfrac{1}{2}(90 - \phi)\left[h' + \frac{h}{2}\right]$$

For the above conditions

$$H = 100 \times 20 \times 0.286 \left(11.09 + \frac{20}{2} \right)$$
$$= 12{,}065 \text{ lb.}$$

Comparing these two results we find that the surcharge sloping at 1½ to 1 produces a pressure about 15% greater than that of uniform surcharge of a height equal to the altitude of the triangle above the top of the wall, in other words greater than that of a Cooper E-50 Engine (four 50,000-lb. axle-loads 5 ft. on centers) spread over a 9 ft. tie, i.e.,

$$w = \frac{50{,}000}{5 \times 9} = 1110 \text{ lb. per sq.ft.}$$

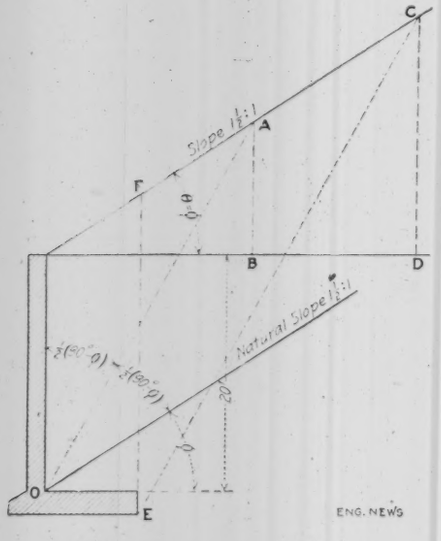
Furthermore if we consider a wall 31.09 ft. (= 20 + 11.09) in height we obtain a total pressure of

$$P = H = \frac{100 \times 31.09^2}{2} \times 0.286 = 13{,}820 \text{ lb.}$$

Still somewhat less than that of the smaller wall.

2. For the design of a slab between counterforts, unit pressures had to be considered.

For the sloping surcharge the pressure at base is



$$p_h = wh \cos^2 \phi = 1385 \text{ lb. per sq.ft.}$$

For the uniform surcharge of 11.09 ft.

$$p_h = w (h' + h) \tan^2 \tfrac{1}{2} (90 - \phi) = 31.09 \times 0.286 = 890 \text{ lb. per sq.ft.}$$

For wall 31.09 ft. high,

$$p_h = wh \tan^2 \tfrac{1}{2} (90 - \phi) = 890 \text{ lb. per sq.ft.}$$

the same as for the uniformly surcharged wall.

This shows the sloping surcharge to give over 50% greater pressure at the base than either of the other two conditions, which seems to be entirely out of proportion. Due to the difference in the points of application of the surcharge pressures the discrepancy between the overturning moments is not so great. For sloping surcharge,

$$M = 13{,}865 \times \frac{20}{3} = 92{,}435 \text{ ft.-lb.}$$

For uniform surcharge,

$$M = 100 \times \frac{20^2}{2} \times 0.286 \left[11.09 + \frac{20}{3} \right]$$
$$= 101{,}565 \text{ ft.-lb.}$$

only a little less than 10% more than for the sloping surcharge.

3. In considering the base, and the maximum soil pressures under it, the discrepancy becomes still more marked.

In this case, where track conditions precluded the possibility of making a larger projection than 1½ ft. at the front, a 7½-ft. projection in back of the wall was assumed. In accordance with common practice for overturning moment, as well as pressures, that height of wall is considered which is obtained by passing a vertical plane through the edge of the base, resulting in a height *FE* in figure, equal to

$$2 + \tfrac{2}{3} \times 7.5 + 20 = 27 \text{ ft.}$$

for which height

$$H = \frac{100 \times 27^2 \times 0.832}{2} = 25{,}280 \text{ lb.}$$

and

$$M = 25{,}280 \times \frac{27}{3} = 227{,}500 \text{ ft.-lb.}$$

For uniform surcharge of height *CD* over wall where *CD* equals

$$5 + \frac{27 \times 0.535}{0.965} = 5 + 15 = 20 \text{ ft.}$$

$$M \text{ (for surcharge)} = 100 \times 20 \times \frac{22^2}{2} \times 0.286$$

$$M \text{ (for fill)} = 100 \times \frac{22^3}{6} \times 0.286$$

and total

$$M = 100 \times \frac{22^2}{2} \times 0.286 \left(20 - \frac{22}{3} \right) =$$
$$189{,}200 \text{ ft.-lb.}$$

from which it would appear that the moment due to the sloping surcharge is almost 25% greater than that due to the uniform surcharge.

4. Another set of conflicting results was obtained in the consideration of a wall 14 ft. high above footing, retaining a fill which sloped from the wall at a slope of 2⅞ horizontal to 1 vertical.

The horizontal pressure for this condition when figured by the formula

$$H = P \cos \theta = \frac{wh^2 \cos^2 \theta}{2} \Big/ \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}}$$

for $\phi = cot^{-1} 1.5$ and $\theta = cot^{-1} 2\tfrac{7}{8}$, is

$$H = 3125$$

For a wall having fill level with top, $H = 2800$ lb., leaving 325 lb. as the pressure due to the surcharge.

For a slope of surcharge equal to angle of repose (1½ to 1), $H = 6785$ lb., leaving 3985 lb. as the pressure due to this surcharge or about 12 times that of a slope somewhat more than half as steep. This is certainly incongruous.

In trying to find some logical method of figuring these pressures and moments the writer thought of applying the method used by Mr. Serber in figuring effects of uniform surcharge, to that of sloping surcharge, that is, to figure weight of wedge and multiply by *tan* ½ (90 − ϕ) to obtain the horizontal pressure.

By this method, for a height of 20 ft. above footing and $\theta = \phi = cot^{-1} 1.5$,

$$H = 100 \times \frac{20^2}{2} \times 0.286 \left(1 - \frac{0.535}{0.965} \right)$$
$$= 8900 \text{ lb.}$$

This result is considerably smaller than that obtained by the other method, but seems to be more consistent, when compared with non-surcharged and uniformly surcharged walls. Figures for other angles of slopes for surcharge are not given (not wishing to take too much much space), but all are consistent with each other and with those obtained for walls, with or without uniform surcharge.

The writer appreciates the fact that this method is at variance with the theory advanced by all the authorities, but even Mr. Rankine admits in his "Applied Mechanics" that the assumptions he makes are very far from existing conditions, but are on the side of safety

M. HIRSCHTHAL.

18 West 112th St.,
New York City, Mar. 4, 1912.

Notes and Querries

Correction—In the diagram on p. 673 of our issue of Apr. 11, illustrating the article entitled "Pitometer Survey of Artesian Wells at Clinton, Iowa," the well marked No. 3 should be marked No. 1, and the one marked No. 4 should be marked No. 3. Well No. 4, as correctly stated in the article, was out of commission and not shown on the diagram.

B. A. Sinn, 60 Broadway, New York City, writes: "When a vessel of steel sinks, there is no wreckage to which those unfortunate enough not to be in the boats can cling, and in this way sustain themselves above the surface of the water for any length of time. In the days of wooden vessels there were hatch covers, gratings, masts, spars, companionways, and other objects on the decks afforded some means of safety; and when the regulations for the amount and size of life boats to be carried on ships were made, this fact was taken into consideration.

"It would seem that a very simple method might be employed, whereby a large number of floating objects could be released when a vessel sinks, by simply putting underneath the life boats, as they are now placed on ships, an equal number of life rafts of large capacity. These life rafts could be handled by the same davits which handle the life boats, and in this way the floating capacity of the life-saving apparatus on board ship could be trebled, with but little expense, small additional space, and very little inconvenience."

Issued Weekly by the
Hill Publishing Company
JOHN A. HILL, Pres. ROB'T MCKEAN, Sec'y.
505 Pearl St., New York.

Monadnock Bldg., Chicago
6 Bouverie Street, London, E. C.
Unter den Linden 71—Berlin, N. W. 7.

CHARLES WHITING BAKER, Editor.

Subscriptions payable in advance, $5.00 a year for 52 numbers, including postage in the United States, Mexico, Cuba, Porto Rico, Hawaii, or the Philippines, $7.00 in Canada.

To foreign countries, including postage, $9.00 or its equivalent, 36 shillings; 36 marks; or 45 francs.

Notice to discontinue should be written to the New York Office in every instance.

Advertising copy should reach New York Office by Thursday of week before date of issue.

Entered at New York Postoffice as mail matter of the second class.

CIRCULATION STATEMENT

During 1911 we printed a total of 931,958 copies.

Of this issue, 18,500.

None sent free regularly, no returns from news companies, no back numbers kept beyond current year.

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Engineering News

Many engineers have become interested in the personality of the late Richard T. Crane, the president and founder of the Crane Co., of Chicago, through his radical criticism of present methods of engineering education. The April number of the *Valve World*, a publication issued by the Crane Co., is devoted to a biography of Mr. Crane. Those who have read Mr. Crane's books, or even his scathing criticisms of higher education in the newspapers, will be well repaid by reading this story of Mr. Crane's life and can form by it a juster estimate of the man and of the attitude from which he viewed the field of education.

According to statistics sent us by the American Association for Highway Improvement, there are in round numbers about 100,000 paid officials actually engaged in the supervision of public roads in the United States. The same authority further states that during the road-building season more than a million dollars a day is spent on road construction and maintenance in the United States, the total expenditures for 1911 aggregating in the neighborhood of $150,000,000. If we assume that the average compensation of these paid road officials during the road-building season amounts to the not extravagant sum of $2 per day, then the total compensation of these paid officials amounts to $200,000 per day, or one-fifth of the entire amount expended on road construction and maintenance. If these statistics put forth by the above named association are anywhere near correct, they point directly toward one chief defect in the present system of road construction and maintenance. Far too many "paid officials" are drawing public money for building (or not building) roads. If the supervision of road construction were placed in the hands of a quarter of the number of the present road officials and these men were paid a salary sufficient to secure competent men, not only would the expense of supervision be reduced, but better roads would be built.

One Sort of Imposition on Engineers Engaged in Municipal Work

No engineer can find fault with the authorities of a city for making thorough inquiry when an engineer is to be selected to carry on a large and important piece of work, in the endeavor to secure a man of high professional standing whose opinion can be safely trusted. It happens not infrequently, however, that city authorities will put the engineers who may be candidates for appointment to large expense by needless delays and by attempting to make one man bid against another. It is needless to say to our readers that the earnings of engineers from the practice of their profession are not so large that losses of this sort can well be borne.

We have received, with the privilege of publication, some correspondence between the Council of a Pennsylvania city of some 25,000 population and a well known engineer, which we print below, omitting names:

Dear Sir—At a Special Meeting of the Sewer Committee I was directed to ask you to appear before that body at a meeting to be held at City Hall, on Friday evening, April 5, at 8 o'clock. As there will be four others to be heard at that time, the conversation will be limited to a half hour. It is suggested you have your subject for discourse condensed so that the time will be well expended. Please advise if you can make your arrangements accordingly, and oblige,

———, City Clerk.

The reply was as follows:

Dear Sir—I have your letter of Mar. 25, asking me to appear before the Sewer Committee at a meeting to be held on Friday, Apr. 5, at 8 o'clock. Your letter also informs me that four other engineers will be heard at that time. I regret to state that I will be unable to be present at the meeting because of an important prior engagement, which I am unable to postpone.

About five years ago I received an invitation from your Sewer Committee to attend a similar meeting for the same purpose, namely, the employment of an engineer to devise a system of sewage disposal for your city, which I accepted. Since that time I have been in receipt of several other invitations of a like nature, and, at considerable loss of time and outlay of money, have each time appeared before your Committee.

Your Committee is advised by a very competent Borough Engineer, and I have stated to him and to the members of your Committee that the course you are pursuing in having these various hearings of engineers is unfair to the various engineers who are put to large expense of time and money, but, also, will, in the end, be without profit to your city. Your engineer is quite familiar with those consulting engineers throughout the country whose standing is well established, and should be able to advise you as to the man you should employ for this work.

The question of compensation is a very minor factor in this matter; yet your chairman advises me that what you are after is to get a competent engineer to do your work for the sum of $500. I do not know whether it will be a matter of gratification for you to know that a fair estimate of the money and value of the time spent by consulting engineers to date in visiting your city at the personal suggestion of your Committee is already over $1000.

Trusting to be able to appear before you at some later date, I have the honor to remain,

Yours very respectfully,

The Demand for Increased Wages to Railway Employees

We said, in our issue of Apr. 11, that if the railways were to grant the demanded increase of wages to locomotive engineers, as well as the demand for a similar rate of increase of wages for other railroad employees, which it may be expected would inevitably follow, the result would be widespread bankruptcy.

It is of interest to see what the official statistics of railway earnings and expenses compiled by the Interstate Commerce Commission have to show on this point. An analysis of these figures made by the Bureau of Railway Economics shows that the amount of increase in railway employees' wages in the year 1911 over the amount they would have received at the rate of wages paid in 1909 was in excess of $70,000,000 for the whole United States. The rate of increase which the locomotive engine runners are asking amounts to an average increase of 18.63% on their present rate of wages. If this rate of increased pay were granted to all railway employees, it would add to the operating expenses of railways in the United States a sum amounting to over $200,000,000 per annum. But the total dividends paid to stockholders by all the railways in the United States in 1909 amounted to only $236,000,000. The rate of increase in railway wages demanded, therefore, would nearly wipe out the entire margin of railway earnings available for dividends.

Inasmuch, however, as a large proportion of railways pay no dividends to their stockholders and are only able now to earn the interest on their bonds, it follows that these railways would at once be placed in bankruptcy. More than this, such a reduction in railway earnings and in the returns on railway capital would make absolutely impossible the financing of railway improvements.

It is generally believed that the country is on the threshold of a year of great prosperity. An increasing volume of railway traffic demands increased facilities in rolling stock and terminals, while safety imperatively demands the extension of block-signal systems and improved appliances for the protection of employees and passengers. All these things are made imperative upon the railway companies by law, but the railway managers cannot make bricks without straw. If returns on railway capital are to be wiped out by the demand of railway employees for higher wages, then a radical increase in freight and passenger rates is inevitable if the railways are to continue in operation.

A British Shipowner's Opinion of the Diesel Engine in Marine Service

Possibly some of our readers may think we were over-enthusiastic in our statement as to the importance of the Diesel oil engine in marine service, made in the editorial in our issue of Apr. 4. Any such, who do not yet comprehend the importance of the oil engine in the future development of ocean transportation, will do well to consider the statement made by Sir Marcus Samuel at the meeting of the Institution of Naval Architects of Great Britain, held in London on Mar. 28. Sir Marcus Samuel is one of the largest shipowners in Great Britain. He was the first to introduce the transportation of petroleum in bulk through the Suez Canal, and has been president of the Institution of Marine Engineers. His firm owns the "Vulcanus," the first ocean-going vessel to be equipped with Diesel engines. Another of the firm's vessels, similarly equipped, will be ready for service this spring.

In the discussion before the Institution, as reported in our contemporary, *The Engineer*, Sir Marcus said that his firm had 70 steamers in service at the present time; but as far as they were concerned they never intended to fit any more steam engines in their ships. In the face of the rapid evolution of the oil engine to its present position, for shipowners to build steamers in future was only courting disaster. From his firm's experience with the "Vulcanus," they were prepared to show that the consumption of oil by this vessel as compared with the consumption of coal by vessels of similar size was as one to five.

Up to the present time, engineers have been considering oil engines for marine service largely on the basis of increased profit they might be able to make by equipping their vessels with the new type of motive power. There is, however, no industry where competition is more keen than in ocean transportation. The great economies possible by equipping vessels with oil engines do not mean in the long run greater profits for shipowners. They mean that under the stress of competition the vessel equipped with steam engines cannot be run at a profit in competition with the oil-engine driven vessel.

It should be recognized, too, that this radical innovation in water transportation may work as great a revolution in inland water transportation as in transportation at sea. It is true that the great advantage gained by the small bulk and weight of fuel necessary to be carried on long voyages does not operate to help the oil engine in inland transportation as it does on the ocean. There still remains, however, the enormous advantage of dispensing entirely with the steam-boiler plant and all handling of coal and ashes and the corresponding great reduction in weight.

The gasoline-motor boat is already coming into considerable use for commercial work. The next step forward, unquestionably, will be the development of motor vessels for inland service which can use oil fuel costing a fourth or a fifth as much as gasoline and free from its inherent dangers.

Economics of Prevention in Hygiene

That "prevention is better than cure" is an old saw, but like many wise old sayings, people are slow to act upon it in actual life. This appears to be true particularly of the comparatively modern science of hygiene and sanitation. This view was presented forcibly by medical and public-health officers at the recent annual meeting of the Lake Michigan Sanitary Association (reported on p. 710 of our issue of Apr. 11) in the course of a discussion on the economics of preventive medicine.

It was pointed out that in spite of the great development in preventive methods, doctors still too often consider that their highest work is in the diagnosis and cure of disease, whereas of greater importance (both from the humanitarian and the economic standpoint) is the prevention of disease. The reason is obvious: cure is uncertain and can be effected in only a percentage of cases. Take, for instance, an epidemic of typhoid fever; cure of every case is impossible; some of those who are cured suffer permanent loss of health or impairment of some faculty; and in every case of cure (perfect or imperfect) there is a loss of time and energy which means an economic loss to the community as well as the individual. How much better, therefore, to guard against disease and prevent its getting a start, thus helping to conserve the economic efficiency of a community.

This, as stated above, applies not only to the mere economic point of view (regarding the individual as a machine) but to the humanitarian point of view; this latter involves not merely the suffering (and death) of individual patients, but the suffering and anxiety (and perhaps privation) of relatives and friends. And these conditions may become practically permanent, as where the patient (whether a wage earner, a business man or a professional man) suffers permanent reduction or entire loss of health and strength. The Chinese custom of paying the doctor to keep his "patients" well, instead of paying him simply to cure them in case of illness, is not such an absurd plan after all. One aim of health and sanitary associations therefore is to impress upon medical practitioners, employers of labor, public officials, and the public at large the immense importance of preventive methods in the field of sanitation and hygiene.

The Greatest of Marine Disasters

In all the annals of shipwreck and marine disaster, one can find nowhere a story of a vessel's loss which has so transfixed with horror the civilized world as has the story of the "Titanic's" wreck. It is not alone that the number who met their death when the great vessel went down far exceeds the number lost in any ocean shipwreck ever recorded. It is because danger and appalling disaster came where they were least expected. The public had come to regard travel in the huge floating palaces, which ply with almost the regularity of ferry-boats between Europe and America, as exempt from the dangers which formerly always beset "those in peril on the sea." The public had come to believe that these modern marvels of marine architecture were not only so huge and strong as to be able to defy all peril of waves and storms, but it had believed that the division of the hull into water-tight compartments, the development of wireless telegraphy, the frequent passage of ships over a regular route, enabling a vessel in difficulty to receive prompt succor, had combined to eliminate all those other elements of risk which, as the records of marine underwriters show only too well, beset ocean traffic under ordinary conditions. It is for this reason that the loss of the "Titanic," with nearly two-thirds of its passengers and crew, a population equal to that of a small city, comes as a great shock to Europe and America.

That the largest and newest vessel afloat in the world, in the construction of which it was supposed that everything had been furnished which ingenuity could suggest or money could purchase to secure absolute safety—that such a vessel should meet disaster in mid-ocean on its maiden trip is proof indisputable that science and engineering have not yet eliminated risk in ocean navigation.

In view of the "Titanic's" fate, it will probably be long before anyone will venture to claim for any vessel that it is "unsinkable." Few engineers, probably, would ever have claimed such a thing for any vessel ever constructed. The only really unsinkable vessel was the old-time wooden-hulled ship which, under certain conditions, after her ballast had gone, would actually float as a derelict.

The division of a vessel's hull into compartments has, of course, saved many a vessel from foundering after a collision which wrecked the bow. In other cases, as in the wreck of the "Republic," it has enabled the vessel to keep afloat long enough to permit the transfer of passengers and crew to another vessel. Even in the case of the ill-fated "Titanic," it kept the vessel afloat long enough to permit lowering her lifeboats. It is well understood, however, by engineers familiar with ship construction that a vessel may have its sides ripped open by a glancing collision with another vessel, or with some other obstruction such as ice, thereby opening several compartments at once to the sea. The same result might follow if the sharp point of a reef or other obstruction in shallow water pierced the bottom when the vessel was under way. Further than this, in any vessel the margin between the normal water line and the top of the hull is such that not many compartments can be permitted to fill before the hull will be plunged so low that water will come over the sides and it will sink at once.

In the testimony of Mr. Ismay, head of the White Star Line, before the U. S. Senate investigation committee, he stated that the "Titanic" was so designed that any two compartments could be filled with water and she would still float, and continued: "I think I am safe in saying that very few ships would float under such conditions."

In the description of the "Mauretania," published in ENGINEERING NEWS some years ago, it was stated that the vessel was designed for safe flotation with two of the largest compartments full. It appears from this that even with the largest vessels, if leaks start in only a few compartments greater than the pumps can handle, the vessel is doomed.

Another subject which is bound to be seriously discussed as a result of the "Titanic's" loss is the relative safety of the vessels of phenomenal size. It may be considered certain that the sinking of the "Titanic," taken in connection with the collision of her sister ship the "Olympic" with the cruiser "Hawke" will have a marked influence in checking the present craze for beating the record in size which has taken the place of the craze twenty years ago for beating the record in speed.

It is well understood that the vessels of record-breaking size which have been built for transatlantic traffic during the last half a dozen years have been profitable not because the cost of ocean traffic decreases with increased size of carrier, as is often claimed, but because the passenger traffic on the north Atlantic route demands and is willing to pay exorbitant prices for an amount of luxury which could not be profitably furnished on any other ocean route in the world. There are a great number of travelers on this route between Europe and America who are willing to pay an extravagant price to travel on the largest and newest ship afloat. Under this stimulus the transatlantic shipping companies have continued to vie with each other in building the largest ships that builders would undertake or harbors would float. The loss of the "Titanic" will at least impress the traveling public with the idea that the size of the vessel is no sure indication of its safety

An illustration of the revulsion of feeling against huge vessels is the story published with great detail in the New York *Times* about the time the "Titanic" sailed from England, to the effect that the North German Lloyd Co. had ordered from the firm of Schichau & Co. a vessel of 54,000 tons register, or 8000 tons in excess of the "Titanic" and "Olympic." Very soon after the news came of the "Titanic's" sinking, this report as to the new North German Lloyd vessel was authoritatively denied.

It should be understood that in our reference to large vessels we are referring to the few vessels afloat of enormous size, in excess of 20,000 to 25,000 tons. There are undoubtedly advantages in steadiness possessed by the vessel of, say, 20,000 tons over a ship of 10,000 to 12,000 tons which mean something to the comfort of passengers in rough weather Experience does not indicate, however, that increasing the size of a vessel beyond some 20,000 tons gives any particular gain in steadiness. It is probable also that when the net profits on the investment are computed, it will be found that the vessel of, say, 20,000 tons yields a higher percentage of net returns, taking the year through, than larger vessels. Indeed, there are those who contend that a 20,000-ton vessel is too large to yield the highest profits even on such a route of dense and high-priced traffic as the north Atlantic. On all other steamship routes it is well known that very few vessels larger than 12,000 tons have yet been put in service. As far as freight traffic alone is concerned, the advantage lies with much smaller vessels, which represent a far smaller investment per ton of carrying capacity. The owner of a freight steamer takes into account the fact that his vessel may at times have to lie idle in seasons when the ocean freight business is dull and realizes the advantage of keeping down the investment in his business.

Undoubtedly one of the chief reasons why the big ships have been so popular is because the public has hitherto supposed that great size in a vessel meant greater safety. Doubtless this fact alone has aided the companies to obtain higher prices for passage on its largest ships. The "Titanic" disaster will probably dispel this notion.

It is doubtful whether even shipowners realize that the larger a vessel is, the more fragile is its outer skin. By fragile we refer not to the stresses from wave action, but to the danger of injury by collision with piers, tugs or other ships, or from grounding. It is true, of course, that the larger vessel has thicker plates and heavier frames, but the thickness is by no means in proportion to the increased size, weight and inertia. If one desires an instructive comparison be-

tween a large vessel and a small one in the matter of withstanding external blows, let him compare the process of bringing a light river or lake steamer up to a pier with the same process where an ocean vessel of 20,000 tons is concerned. For a still more convincing comparison, read Commander Peary's account of the voyage of the "Roosevelt" through the Arctic ice in Baffin Bay, Smith's Sound and Kennedy Channel to the station from which he made his final successful dash to the Pole. Of course it is true that the "Roosevelt" was a ship especially built to withstand ice pressure and collision with floating ice; but had she displaced as many thousands of tons as she did hundreds, her inertia would have been so great that she never could have withstood the butting against the Arctic ice.

In the "Titanic's" collision with the floating ice mass which sent her to the bottom, there are probably few outside the engineering profession who realize that the pressure produced against the vessel by collision with any stationary object increases directly with the tonnage of the vessel.

In the discussion of the loss of life in the "Titanic" disaster, public attention has been concentrated chiefly on the lack of life-boats sufficient to carry all the people on board the vessel. All the shipping companies are being severely censured because they have not furnished life-boats on their vessels in proportion to the number of passengers and crew carried. It is fair to say at this time, however, that the White Star company has simply followed common practice in this matter. There are no large passenger vessels afloat today, that we know of, either in inland waters or on the ocean which carry boats enough to rescue the entire ship company in the event of the vessel's loss.

Of course at this time, with the news of the "Titanic" disaster fresh in everyone's mind, a universal demand goes up that every vessel must carry life-boats sufficient to accommodate every person on board. It is well to reflect, however, that even if this provision were carried out, it would be very far from insuring safety. It must be remembered that after the "Titanic's" collision the circumstances were unusually favorable for launching her life-boats. The sea was phenomenally calm, the vessel's lights were all burning and the serious injuries the vessel had suffered were hardly realized by the passengers, such was their confidence in the vessel's stability, until the end was very near. Undoubtedly this had much to do in averting panic and its disastrous consequences.

There are wrecks, however, where a vessel goes down too suddenly to give time for launching the boats. In the "Bourgogne" disaster the stricken vessel after collision listed so much as to prevent most of the boats being handled. Again, during a large proportion of the time the north Atlantic is too rough to permit life-boats to be launched from a vessel and passengers to be placed therein without heavy risk of swamping or smashing the boat against the vessel's sides. Much of the time the sea is too rough for open boats to long survive even when safely launched.

Even with the favorable circumstances on board the "Titanic," absence of panic, good discipline of the crew, and ample time for lowering and loading, some of the boats went off with only a few in and others crowded to the limit of their capacity. Had all boats been as full as these latter, nearly 200 more persons might have been saved. Yet no better results in systematic loading could be reasonably expected on a vessel about to founder.

We point out these facts, not to oppose the public demand that more adequate provision be made in the way of life-boats, life-rafts, etc., to secure safety at sea, but in order that attention may not be unduly concentrated on this detail to the exclusion of matters of equal or greater importance. Life-boats are, at best only a last resort.

Up to the time when the "Carpathia" arrived in New York with the "Titanic's" survivors, it has been generally assumed that the unfortunate vessel collided with the iceberg while sailing through a fog. From the accounts of the survivors, however, it appears that the night was a clear one, although dark, and that the navigating officers failed to see the ice mass in their path until too late to steer clear of it. There has been more or less loose talk of the vessel running onto a submerbed iceberg, as if it were possible for ice to float waterlogged barely showing above the water surface. On the other hand, it is a fact that a floating block of solid ice has only 8½% of its bulk above the water level. It is, therefore, quite possible for a block of ice with a bulk of many tons, sufficient to badly strain the plates of a rapidly moving steamer coming in contact with it, to float with so small a portion of its mass exposed above the water that the lookout from the bridge, or the crow's nest, of a rapidly moving vessel might readily fail to see it on a dark night.

It must be remembered that no very serious shock against a vessel's side or bottom, comparatively speaking, is necessary to open the seams and start leaks too great for the pumps to keep pace with. Should a glancing blow be struck, springing the plates and seams and admitting the sea to four or five compartments, the vessel may be doomed even if the bulkheads themselves remain perpectly tight.

It appears to us, therefore, that the first and most important lessen of the "Titanic's" loss is the need for moving the summer course of transatlantic steamers between North Atlantic ports and English Channel ports farther south. This terrible disaster shows that the present southern course, which is followed from Jan. 15 to Aug. 23, is not far enough south to avoid ice dangers in all seasons. To guard against the ice danger now existing in the usual track of vessels, a new transatlantic course has already been laid down, as reported elsewhere in this issue.

Of course it will not be necessary to follow this southern and much longer course in ordinary seasons; but what should be done, it seems to us, is to provide timely warning to vessels of the approach of icebergs to the transatlantic track. Instead of waiting until the floating ice masses actually reach the transatlantic track and endanger shipping, it would be practicable to have a cruiser of the revenue marine service dispatched to northern waters as early in the season as the arctic current brings its ice burden down as far as the latitude of Newfoundland. Such scouts could give seasonable warning to vessels of the approach of ice to the ocean track and the transatlantic routes could be shifted as the news of such ice movement demanded.

It must be emphatically impressed upon the masters of all vessels that floating masses of solid ice may make little show above the water surface and still be capable of seriously injuring the hull of a vessel colliding with them at high speed.

Finally, although the loss of the "Titanic" as now reported seems to have occurred in clear weather, it is to be hoped that this greatest of marine disasters will not pass by without some concerted effort to stop the present practice of running at high speed in dense fog. Let it not be forgotten that the "Republic" disaster, only a few years ago, when such a loss of life as has now occurred was escaped only by a hair's breadth, was due to a collision in a dense fog. Wireless telegraphy has, it is true, somewhat reduced the dangers of navigation in a fog; but there are thousands of craft afloat not equipped with wireless and hardly a week passes but brings its record of marine disasters due to collision in fog.

It is probable that some of the recklessness with which steamship companies and vessel masters allow their vessels to plunge ahead at practically full speed when the fog is so thick that one cannot see half a ship's length ahead has been due to misplaced confidence in the stanchness of the vessel's hull and to the idea that modern huge high-class vessels were practically unsinkable. The "Titanic" disaster definitely ends this fallacious notion. If it shall lead to the cutting down of a few knots in the speed with which ocean vessels are propelled through dense fog, the lives lost on the "Titanic" will not have been lost in vain.

The Wreck of the "Titanic:" Its Effect on Transatlantic Steamship Routes

One immediate result of the sinking of the White Star liner "Titanic," after collision with an iceberg on Apr. 14, has been the adoption of new routes for transatlantic steamships some 200 miles south of the former summer routes. The positions of the old and new routes are indicated on the accompanying chart.

The official statement of the disaster given out Apr. 21 by the White Star Line is as follows:

"Titanic" followed strictly southernmost track westbound, changing course at corner 47 meridian 42 latitude, thence south 86° west. True all officers watch perished, except Fourth Officer Boxhall, who was working observations in chartroom and making rounds. Night perfectly clear starlight, no wind, sea calm. Had encountered no ice previously. Proceeding with vigilant lookout full speed, but reduced consumption, probably 21 to 22 knots. Engineers all perished. 11:45 p.m., Apr. 14, ship sighted low lying berg direct ahead. First Officer starboard helm reversed full speed, and closed all compartments. Struck berg bluff starboard bow. Slight jar, but grinding sound, evidently opening several compartments starboard side. Boats cleared, filled with women and children, lowered and sent off under responsible persons. Ship sank bow first 2:20 a.m. All boats away except one collapsible. "Carpathia" rescued 4 a.m. Discipline perfect.

The time of the accident was given in our issue of Apr. 18 as 10:25 p.m. This, however, was the time when the first news of the wreck was received at the Cape Race wireless station. The difference in time between this station and the place of the wreck accounts for the discrepancy. It appears then that the vessel went down about 2½ hours after the collision.

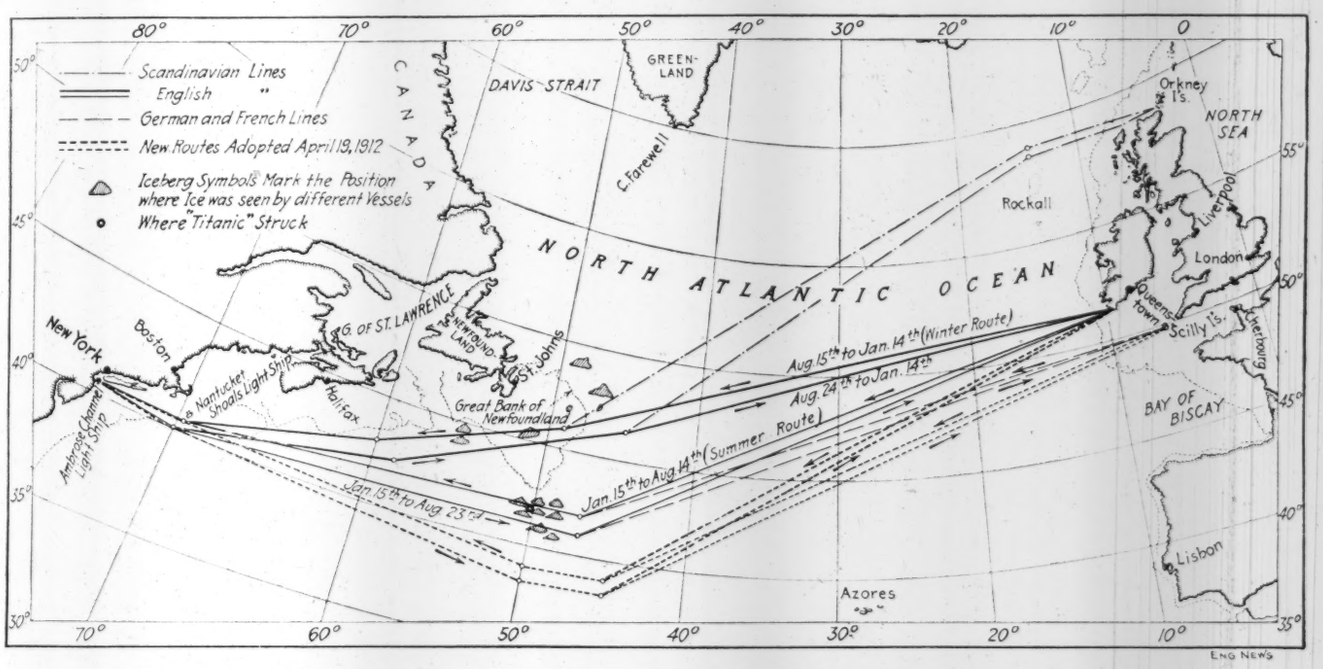
From the accounts of survivors, as published in the daily press, it is fairly well established that the ship sank bow first and slowly to the very last. The inclination of the deck toward the bow gradually increased and the stern rose from the water, several observers who left the ship near the last noticing that the propellers were visible. Nothing like a rapid plunge seems to have occurred until only a portion of the hull, at the stern, remained above water. The "suction" of the sinking ship was much less than was expected.

The lighting system, according to the stories of survivors again, remained in operation practically until the ship went down, and this was a most important factor in aiding the work of loading and launching the lifeboats. An explosion of considerable violence is mentioned in the stories of a number of the survivors, some of them even believing that the vessel was broken in two near the middle just before she sank.

The steamships "Carpathia," "Virginian," "Californian," "Frankfurt," "Birma," "Baltic" and "Olympic" were informed of the disaster by wireless and started toward the sinking "Titanic." The "Carpathia" was the first to reach the scene, at about 4 a.m., and picked up the survivors from the "Titanic's" lifeboats. A statement issued by the North German Lloyd S. S. Co., Apr. 21, gives the time of arrival of the "Frankfurt" at the scene of the disaster as 10:50 a.m., and adds that she found there the steamers "Birma," "Virginian" and "Carpathia."

The number of lifeboats carried by the "Titanic," according to the testimony of Fourth Officer Boxhall before the Senate investigating committee, was 14. In addition there were two "sea boats" and four collapsible boats. The capacity of the lifeboats was 60 passengers each and the other boats could each carry about 25 or 30. Thus the total capacity of all boats was about 1000.

The survivors, all of whom were brought to New York on the "Carpathia," numbered 705. The number of lives lost, including those who died from exposure in the lifeboats, was 1442. Of those saved, 210 were members of the crew and 846 of the dead were passengers. This is according to figures given out to the press Apr. 19 and 20 by W. W. Jeffries, General Passenger Agent of the White Star Line.



GREAT CIRCLE CHART OF THE NORTH ATLANTIC, SHOWING THE NEW SUMMER ROUTES FOR STEAMSHIPS

(Winter routes of German and French lines not shown. The turning points are the same as for the English lines. These points are indicated by small open circles.)

THE NEW ROUTES

The position of the "Titanic" as given in her calls for assistance by wireless, was lat. 41° 46′, long. 50° 14′. It will be seen by referring to the accompanying chart that this was 14′, or about 16 miles, south of the regular westbound summer route. The early reports that the "Titanic" was using the shorter route —the northern or winter route—were plainly erroneous.

The general position of the group of icebergs upon one of which the liner was wrecked is indicated on our chart. The iceberg symbols on the chart mark posi-

tions at which icebergs were sighted by different vessels in the few days immediately preceding and following the wreck. For the sake of clearness, only a few of the numerous reports received at the New York office of the U. S. Hydrographic Bureau are marked on the chart.

The course of the group of bergs to the southward under the influence of the Labrador current can be traced by means of the pilot charts issued monthly by the Hydrographic Office. The Labrador current curves around the coast of Newfoundland and is believed to pass below the Gulf Stream. The ice brought down by the current from the north is picked up by the Gulf Stream and carried slowly eastward, being at the same time rapidly melted as a general rule by the warmer water. The speed of the Gulf Stream in this vicinity is only 15 or 20 miles per day. Slight variations in the flow of the two currents or a difference in the character of the ice itself may account for the further progress of the ice to the southward in some years.

The progress of the ice southward was indicated in another way by the nature of the ice reports received at the Hydrographic Office from steamships sailing over the regular summer routes, which are used from mid-January to the middle of August. Field ice was reported as early as Mar. 29 and 30, some steamers finding it advisable to take a more southerly course to pass around it. Beginning with Apr. 3, bergs of considerable height were reported in increasing numbers, and from Apr. 10 to 20 as many as 30 or 40 separate reports of ice bergs came in. Typical reports for this period stated that "a large number," "two hundred" and "several hundred bergs" were sighted.

The ice this year is further south than it has been during a long period of years, and on Apr. 16 the transatlantic lines announced an agreement "in consequence of the reports as to ice in the Atlantic" to shift the established routes so as to bring them 60 or 70 miles farther south in the vicinity where icebergs are met. Three days later, on Apr. 19, an agreement was effected between the steamship lines and the U. S. Hydrographic Office moving the routes some 100 miles farther south still to the position indicated on the chart reproduced herewith. The chart shows also the former routes, which were established in 1898. The new routes are about 175 miles longer than the former summer routes.

We are indebted to Lieut. John Grady, U. S. N., in charge of the New York branch of the Hydrographic Office, for assistance in preparing the chart.

The Sinking of a Steamship at Sea

The accompanying cuts represent very graphically the sinking of a vessel at sea which went down in almost identically the manner described by those survivors of the "Titanic" disaster who witnessed the final plunge of that great vessel.

The cuts are reproduced from ENGINEERING NEWS of July 14, 1898. Curiously enough, the vessel whose last moments afloat are there depicted was lost, like the "Titanic," on her maiden voyage. It is probably not generally known that extra insurance is often taken out to cover the first voyage only of a vessel and is charged for by the underwriters at a higher rate to cover what is believed to be the extra hazard due to a new vessel, which has not yet "found herself," as Kipling says, and with new engines and a newly organized crew.

We reprint below from our issue of the above date the description then given of the vessel's sinking:

The "Tai Hoku" was a steamer of 3100 tons, built at Middleboro, England, for the Osaka Steam Navigation Co. Having sailed to Antwerp to complete her cargo, the steamer left that port on Aug. 6, 1897, for Japan. On the following Sunday she passed into a heavy fog, off Cape Raso, and at about 9 p.m. the steamer collided violently with another vessel. At first it was supposed that no serious damage had been done, but in half an hour 5 ft. of water was reported in the forehold. The water gained at the rate of about 2 ft. per hour, and the ship gradually settled by the head.

At 4 a.m., on Monday, there were 16 ft. of water in the forehold, and all the pumping power available made no impression upon the inflow. At 5 a.m. all hands took to the boats and pushed off a few hundred yards from the sinking steamer, and at 8:30 a.m. the party was rescued by the steamer "Millfield." By this time the fog had cleared away, and the stern of the "Tai Hoku" could be seen gradually rising in the air. Shortly before 1 p.m. she began to roll heavily and then she plunged and sunk. In the 15 minutes previous to her final disappearance four photographs were taken, of which three are here reproduced.

When the "Tai Hoku" went down, there was, of course, considerable steam pressure in her boilers, possibly 60 or 70 lb. When the bridge collapsed, the siren and fog-horn cords being attached to it, both the siren and fog-horn sounded, just as the vessel disappeared. The third and final view shows, apparently, an explosion. But, as a matter of fact, said an engineer who witnessed the accident and took the photographs, there is little doubt that when the water reached the boiler flues there was a sudden generation of steam and gas, which rushed with almost an explosive effect up the flues and smokestack, carrying with it quantities of soot which remained suspended in the air for some minutes after the ship disappeared. There was not the least sign of the proverbial whirlpool when the ship went down, though there were abundant surface signs of the gradual escape of the air imprisoned in the hull.

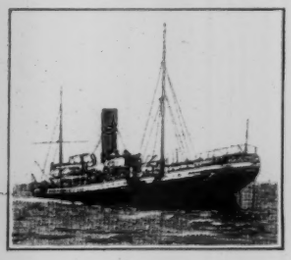


FIG. 1. FIFTEEN MINUTES BEFORE SINKING

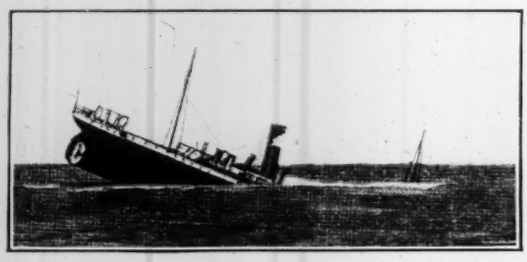


FIG. 2. FIVE MINUTES BEFORE SINKING

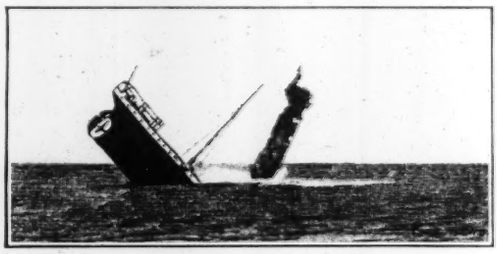


FIG. 3. THE FINAL PLUNGE

THE SINKING OF THE STEAMER "TAI HOKU" OFF CAPE PASO, FRANCE, AUGUST, 1897

(Reproduced from Engineering News of July 14, 1898.)

An Important Potash Deposit has been discovered at Borax Lake in the northwestern corner of San Bernardino Co., Calif., according to a bulletin issued jointly by the U. S. Geological Survey and the Bureau of Soils. Borax Lake, or Searles Lake, is one of the many playas or intermittently wet and dry lakes common throughout the arid regions of the West. It lies between the Argus and Slate ranges in the Mohave Desert and was the original scene of the famous 20-mule team Borax mine. The lake is about ten miles long and five miles wide. According to the bulletin, brine samples taken from six wells distributed over the salt flat show an average of 6.78% of potassium oxide (K_2O) in solution. The average salinity of the brine is 43.82 grams of solids per 100 cu.cm. Existing data give reasonable assurance that the brine-saturated salt body is at least 60 ft. thick, and covers an area of at least 11 square miles. The total amount of potassium oxide is conservatively estimated at more than 4,000,000 short tons. Methods of separating potash from brine are now under investigation by the Bureau of Soils.

The Present Position of Wireless Telegraphy*

By Count George von Arco†

The most important technical progress made by wireless telegraphy in its 14 years development is due to the laboratory work of the large firms engaged.

The two undertakings which control its development almost exclusively are the Marconi company in England and the Gesellschaft für drahtlose Telegraphie in Germany. The former took the lead during the first 10 years of development, while the latter has come to the fore in the last few years.‡

Prof. Slaby was stimulated to new ideas of his own after witnessing the first Marconi trials in the year 1897. He designed an independent wireless system which was worked out in the Oberschöneweide cable works of the Allgemeine Elektricitäts Gesellschaft. Almost simultaneously Prof. Braun, of Strassburg, secured a number of fundamental German patents, the use of which he transferred to the firm Siemens & Halske. In the year 1903 the two large firms founded the Gesellschaft für drahtlose Telegraphie m. b. H. for the common exploitation of these patents. The name "Telefunken" was selected for the system. The operations of the new company were at first restricted to the delivery and installation of wireless stations principally for military purposes.

Marconi's inventions were exploited by an English company which undertook both the delivery and installation of plants, as well as the erection of wireless stations for the transmission of general news. The favorable geographical and political position of England was very advantageous for this purpose. The organization of the English company for the transmission of news was already very extensive and well established, and an English Marconi world monopoly for the wireless transmission of news was half completed when the German company decided to extend its activity to this branch also.

After the Telefunken company had installed stations on some of the ships of the German mercantile fleet, the Marconi company refused a mutual communication on being called up by German ships. On this account the German merchant service was compelled to equip all its ships with Marconi stations, and to include the telegraphists of this foreign organization in their crews. In view of the great interest which the German state authorities and shipping circles had in putting an end to these conditions, the Allgemeine and Siemens & Halske concerns determined to come to an agreement with the Marconi company at a considerable financial sacrifice to themselves, by which the wireless service on board German mercantile ships was taken over by a newly formed company, "Deutsche Betriebsgesellschaft für drahtlose Telegraphie m. b. H." This company has the right of employing the German Telefunken and Marconi patents, and the 160 ship stations which it operates at the present day enjoy the same rights as the remaining Marconi stations. Thus at the present time the German company, including both the supply and operating companies, takes part in the working of the wireless system.

The number of stations installed is a guide to the importance of the different firms in the markets of the world. The office at Berne, in its official list for the year 1910, mentions some 1300 wireless stations which are distributed throughout all parts of the world. Of these, 80 to 85% are registered as employing the Marconi and Telefunken systems, the use of the two systems being about equally divided.

The principal apparatus in every wireless installation is (1) the transmitting apparatus for producing high frequency alternating currents, (2) an antenna for radiating this energy, (3) a second antenna at a distant point for collecting the incoming radiations, and (4) a receiver which makes them perceptible.

The alternating currents may take two different forms: (1) undamped or (2) damped wave-trains. The first are produced either in accordance with Poulsen's method by an arc in an atmosphere of hydrogen, or directly by means of an alternating-current dynamo specially built for this purpose, a so called high-frequency machine, and the latter are produced by a spark discharge. The arc method has not fulfilled the hopes placed in it and its application in practice has remained very limited.

The continuous oscillations have only one single periodicity, the number of oscillations per second; the intermittent oscillations, on the other hand, have, in addition to the number of oscillations, a periodicity of the groups. As each group is derived from the energy of one spark discharge, the periodicity of the groups is equal to the rate of sparking per second. A transmitter with intermittent oscillations can, therefore, possess more individuality.

As already mentioned, frequencies up to 1,000,000 periods per second are adopted for high-frequency currents in our technique. Such frequencies cannot be produced either at the present time or at any early period in the future by a high-frequency dynamo. Alternating currents of 50,000 periods have, however, also acquired a certain value in the last few years, particularly for telegraphing over very great distances. Such frequencies can be generated directly on the dynamo principle from mechanical energy, by means of certain special dynamos.*

The high-frequency machine is expected to produce an increased wave energy. Hitherto, however, no failure in wireless telegraphy has been due to the fact that insufficient quantities of energy were available for the transmitter, but because they could not be radiated. The antenna difficulties limit the outputs. When antennæ have actually been invented by means of which 500 or 1000 kw. can be radiated, then perhaps the spark method will fail, and then the machine may assume a greater importance.

High-frequency currents up to more than 100 kw. and frequencies up to millions per second, and down to a few thousands, are produced with spark discharges. The most modern spark method can be well demonstrated by means of a tuning fork which is struck. The energy is imparted to the tuning fork by a short stroke, and this energy is converted into an acoustic wave-train of decreasing amplitude, and radiated. When the tone has entirely or nearly died away a new stroke is given. The hammer strokes correspond to the sparks in the electrical simile. The quantity of energy of a spark is converted into an alternating-current wave train of decreasing amplitude.

In German stations the sequence of the strokes is usually at the rate of 1000 per sec. Let us assume that the alternating current generated has 100,000 periods, and that each wave train ceases after 100 oscillations, then the intervals between the wave trains exactly disappear and the new spark always occurs at the moment when the previous wave train ceases. The spark method has some advantages over the machine. Of these we only mention the absolute constancy of the periodicity which is here dependent upon fixed electrical values, the double characteristic of the transmitter for high and tone frequency, and the variable accumulation of the energy per second for obtaining greater momentary effects at the receiver.

The most perfect form of the spark method at the present day is the "sounding blown-out spark" system. Three electrical properties should be mentioned in particular: (1) the intervals between the wave trains are extremely short, (2) the distances between the groups are of equal length and the wave trains follow one another with absolute regularity, whereby a tone is produced in the receiver telephone; (3) the rapidly extinguished spark has the further advantage

*From a paper reprinted in the "Allgemeine Elektricitäts Gesellschaft Journal," February, 1912.

†Deutsche Betriebsgesellschaft für drahtlose Telegraphie, Berlin, Germany.

‡It is now announced that the United Wireless Co., controlling the American commercial service, has been consolidated with the Marconi company.—Ed.

*See note on "wired wireless" for multiplex telephony and telegraphy as arranged by Maj. G. O. Squier, U. S. A., in Eng. News, Feb. 2, 1911, p. 152, and also a note on the next page on use of high-frequency currents in military telegraphy.—Ed.

that it exists only during the very first oscillations, a long aërial wave train continuing after its extinction. The energy loss of the spark is confined to a very small fraction of time, or speaking practically, it is entirely overcome. After a long period of laboratory work, this extinguishing principle, given by Prof. Max Wien, has been developed by the Telefunken company to a stage of perfect reliability, even in plants working with a wave energy of 100 kw.

The available high-frequency energy is led to the antenna by the transmitter, a part being radiated from it and producing a distant effect. This is the actual work of the antenna. Only very recently has more light been thrown upon the phenomena occurring in the antenna, and the distance effects. Within the last two years Dr. Kiebitz, engineer in the telegraph experimental department, has carried out new experiments with improved earth devices, and has published the results, some of which are very favorable. Among other matters, he states that by means of an antenna erected by five workmen in one morning he was able to hear the signals of a station in Canada at a distance of 3700 miles.

The form of antenna hitherto employed generally consisted of wires with an upper conducting end surface, which were led vertically upward and supported by high masts; the dimensions were dependent upon the distances and the quantities of energy. The greater the distance to be bridged over, the greater must the energy be which is sent into the antenna and radiated from it. The greater the energy is, the larger must the antenna be, both as regards the upper wire surface and the height of the same. The cost of the towers increases almost as the cube of their height. Here, then, lies the practical limit for the transmission range of wireless transmitting stations. It is certainly possible to generate 100 or even more kilowatts in the form of high-frequency alternating current; one cannot, however, build without enormous expense an antenna which will radiate it satisfactorily. If the earth antennæ fulfill all the hopes which many experts promise for them, then perhaps a new epoch in the construction of large wireless stations will begin for the longest distances ever known to our world, and then perhaps the high-frequency generator will receive preference over the spark system. One must not be too optimistic, however.

The same antennæ are used for receiving as for transmitting. By syntonizing the receivers greater distances can be covered on the one hand, and on the other hand it is possible to receive the signals of other transmitters, or not to hear them, according to the adjustment. How are the weak currents of the receiver antenna made perceptible? The telephone cannot be operated by them, as its diaphragm is much too slow to follow the 100,000 or more periods of the receiving current. The acoustic organ of the human ear is also too slow and would not hear these telephone tones. It is therefore necessary to convert the energy. The high-frequency alternating current is converted to continuous current. This is effected by means of a rectifier, the detector, which at the present day usually consists of a metal point in contact with a special mineral, for example, platinum and lead sulphide.

From a damped high-frequency wave train a continuous current impulse is produced by rectification, and the latter finally moves the membrane of the receiver. A movement of the membrane takes place for every distant spark. At 1000 sparks per second the tone with 1000 vibration periods at the receiver is heard.

The technical progress of the last few years can best be realized from the extended range of application. The transmission distances of the stations have been relatively increased, as it is now possible to transform 50 to 75% of the machine energy into antenna energy. Requirements can, therefore, be met with smaller primary plants. Owing to the omission of intervals between the wave trains, more energy can be led to one and the same antenna, and owing to the short duration of the spark, it is possible to transform very large quantities of energy into vibrations without disturbing the spark electrodes. The freedom from disturbances caused by other stations and atmospheric discharges has also been increased to an extraordinary extent.

The construction of all apparatus differs fundamentally according to whether it is required for military purposes or for the transmission of general news. In case of war it is necessary to get one's telegrams through, notwithstanding intentional disturbances caused by the enemy. A large selection and a large tone scale is required, and it must be possible to make all alterations in the electrical adjustment as quickly as possible. Complications and high prices for the apparatus are the natural result.

The apparatus for the mercantile fleet is much simpler and smaller, since as a rule only short distances come into question.

When one hears that with these small stations, which send 1.5 kw. into the antenna, it is possible to bridge over distances of many thousand kilometers, one would think that with an antenna energy of 35 kw. it would be possible to reach fantastic figures. This is not the case, however. These long-distance results on ships are obtained at night. During the day, on the other hand, only distances of 600 to 700 km. can be covered.

Marconi was the first to recognize the cause. Light is the enemy of electric waves, and a greater enemy and energy disturber, the higher the frequency of the alternating current. It is certainly easy to produce low frequencies but very difficult to radiate them economically. The antenna determines the maximum frequency, by its height. The higher an antenna is, the better it is for low frequency. Thus we are here faced by a great difficulty. If it is desired to make a permanent connection over a long distance, a connection which will work in the strongest sunlight, as for example, at midday in the tropics, then a low frequency is necessary. For this purpose a very high antenna is required. Marconi was also the first to establish a permanent connection over 1900 miles between England and Canada, which is still the only plant of the kind. His two stations are both the same size, and are both provided with enormous antennæ.

Use of High-Frequency Currents for military telegraphy is described in a recent paper by Maj. G. O. Squier, U. S. A., before the Franklin Institute of Philadelphia. The experimental field lines employed were of standard Signal Corps field wire, with 11 strands of steel and one strand of copper, weighing about 70 lb. per mile, and having a resistance of about 11 ohms per mile. The generating apparatus consisted of small portable wireless sets furnished by the Signal Corps for field use. These include a 500-cycle generator with a special transformer and a quenched-spark gap for the production of a continuous train of oscillation. The spark gap consists of built-up disks of copper separated by layers of mica ring insulation.

The standard field wire furnished by the Signal Corps when laid out on the ground could be used to transmit Morse signals by connecting directly or through the open-core transformer to the 500-cycle generator supplied with the wireless equipment, and the signals could be interpreted at the receiving end by a standard low-resistance telephone receiver. The same wire could be used for transmitting the high-frequency discharges of the complete quenched-spark field sending set which were received by the complete receiving set of such a wireless apparatus (with the usual detector and high-resistance telephone receivers). Since the high-frequency oscillations used in the last case were above the limit of audibility, it was necessary to insert an interrupter to break the wave trains up into groups, which gave impulses within the limits of audibility. The quenched-gap circuit may be coupled directly to the line but then, since the line is of comparatively high resistance and small inductance, the waves are damped out too quickly. This is cured by the use of an intermediate circuit of low resistance and high inductance. The interrupter may be placed in the generator or receiving circuit. It is better placed in the receiving circuit since the signals then sent out along the field wire are entirely inaudible and can be interpreted only with apparatus which includes an interrupter. The transmission of the regular Morse signals over the wire with the usual field buzzer apparatus is not interfered with by simultaneous use of the "wired-wireless" as noted.

The National Waterways Commission

The final report of the National Waterways Commission just sent to Congress shows that traffic on inland waterways has diminished since 1909. It recommends that the Government furnish free engineering for the construction of the Lake Erie & Ohio River ship canal if the localities interested raise the money to build it. It further recommends that the Interstate Commerce Commission be given authority over waterway transportation; that no Federal money should be spent on waterways unless terminals free from railway control shall be assured; that the Government should share the cost of storage reservoirs to prevent Pittsburg floods; and that water-power development should be subject to Federal control.

The report has just been made public of the United States Waterways Commission, created by act of Congress, Mar. 3, 1909. The preliminary report of the Commission was reviewed in ENGINEERING NEWS of Feb. 3, 1910, p. 137. The Commission is made up of seven Senators and five members of the House of Representatives. The chairman of the Commission is Hon. Theodore E. Burton, of Ohio, whose long service as chairman of the Rivers and Harbors Committee of the House of Representatives gives him such an intimate knowledge of the government work in connection with river and harbor improvement as is possessed by very few public men.

The complete report forms a pamphlet of 62 pages. The Commission has also had printed some 22 documents of special interest to students of transportation problems, including reports on railways and waterway transportation in foreign countries. In what follows we present a summary of the conclusions and recommendations contained in the Commission's final report.

Is River Commerce Increasing or Diminishing?

In its preliminary report the Commission referred to the decline in water transportation on many inland rivers, especially the Mississippi and its tributaries. To determine whether any change in conditions has occurred since the date of that preliminary report, the Commission obtained from the Chief of Engineers reports from 44 officers in charge of districts, showing the present status of river commerce in each district. In the majority of cases the reports show that the traffic has remained practically stationary since 1909. There has been some increase in traffic on the rivers emptying into the Atlantic Ocean which are large enough to accommodate ocean or coast-wise vessels. These, however, are really arms of the sea and are not to be classed with inland transportation.

On the Mississippi and its tributaries, while there have been some cases where traffic has increased, the result generally has been decrease. An increase on the Tennessee and Cumberland Rivers appears to be chiefly due to lumber traffic. On the upper Mississippi River, between St. Paul and the mouth of the Missouri, a traffic of nearly 4,000,000 tons was carried in 1907. In 1911 the traffic was little over 2,000,000 tons. Log rafts, which formerly furnished a large part of the traffic on this section of the river, are disappearing with the cutting off of the timber.

The great amount of agitation over waterway transportation has led to the establishment of a steamboat line on the Missouri River by Kansas City capital, which is to begin operations this year. On the Mississippi River a barge service between St. Louis and New Orleans, which was discontinued in 1903, was re-established last June, but its total traffic for the season was only 11,130 tons. There is considerable activity in the construction of steel barges for traffic on the Ohio River.

The Projected Lake Erie and Ohio River Canal

A year ago the Commission was authorized by Congress to investigate this enterprise. At the Commission's request, a report upon the project has been prepared by Col. H. G. Newcomer, Corps of Engineers, U. S. A., and has been published by the Commission. The proposed canal is to extend from the Ohio River, just above Dam No. 6, up the Beaver and Mahoning Rivers to Niles, Ohio, a distance of 50 miles, and thence up the Mosquito Valley to the summit level near Jefferson, Ohio, which is 31 miles long. The canal then descends with a vertical fall of 327 ft. through Indian Creek Valley to Lake Erie, a distance of 13 miles. The total length of the canal would be 103 miles, and the total elevation overcome 559 ft., for which 30 locks would be required. The canal is planned for a depth of 12 ft. over miter-sills. Surveys made in 1905 indicated a total cost of construction of $53,000,000. The Commission says it is now generally conceded that that estimate was somewhat too low and that $60,000,000 would be more nearly correct. The Commission gives its chief attention to the question whether the traffic the proposed canal would obtain and the benefits from its construction would be commensurate with its cost. On this point we quote from the report verbatim as follows:

> The calculations made in 1905 give 3,000,000 tons as the probable traffic for the first year of operation, 22,500,000 for the fifth year, and 38,000,000 for the 10th year. There are a number of considerations which would indicate that these estimates are probably too high. In order to successfully compete with the railways the canal must offer much cheaper transportation, except when there is an excess of traffic. This is due to the fact, as set forth in the preliminary report of the Commission, that water transportation has certain natural disadvantages which lessen its convenience and reliability, and cause shippers to patronize the railway in preference, unless the water route offers sufficient inducement in cheaper transportation.
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> The cost of transportation on the proposed canal is estimated at 1.58 mills per ton-mile, which, with the tolls proposed to be charged, would make the total cost about 3 mills per ton-mile. This would undoubtedly give the canal the required advantage over the competing railways if the present rates, which are generally considered to be high, amounting to about 8.5 mills per ton-mile for iron ore and 6.8 mills per ton-mile for coal, were maintained after the canal was opened. It may confidently be expected, however, that the railroads will make substantial reductions from the present high rates whenever they are not operating to their full capacity, in an effort to keep traffic from being diverted to the waterway, and every reduction in their rates will lessen the advantages of the canal.
>
> Some idea of the point below which the railroads could not profitably reduce their rates is shown by the fact that the cost of shipping ore on the Bessemer & Lake Erie R.R., which is owned by the United States Steel Corporation and operated as a bulk freight road from Pittsburg to Conneaut on the Lake, is stated to be about 2.8 mills per ton-mile, and the cost of hauling coal in full train loads on the Pittsburg & Lake Erie R.R., which the canal would parallel for most of its length, according to estimates made by Frank Lyon, attorney for the Interstate Commerce Commission, is less than 2 mills per ton-mile.
>
> Several cases involving the fairness of these coal rates from the Pittsburg district to the Lake Erie ports have recently been considered by the Interstate Commerce Commission and reductions amounting to 10c. a ton ordered. The complaint investigated was that these rates were unreasonably high in comparison with the West Virginia coal rates, some of which average less than 3 mills per ton-mile.
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> The railways competing with the proposed canal will doubtless be restrained from making excessive reductions in their rates by the provision of the Mann-Elkins Act of 1910, which prohibits railroads lowering their rates in competition with a waterway from raising them

again until after hearing by the Interstate Commerce Commission. This provision was inserted in accordance with the recommendation of the National Waterways Commission in its preliminary report, the purpose of which was to prevent the elimination of water competition by unfair means.

It is the opinion of the Commission that the amount of coal which it is expected the proposed canal would carry has been overestimated. The coal deposits in the Monongahela Basin near to the river are becoming depleted so that it will be more expensive in the future to bring to the river the coal that is to be shipped in barges. This would make it more economical, as well as convenient, to patronize a railroad whose tracks reach directly to the mine.

To what extent these factors and the natural disadvantages peculiar to waterways will reduce the amount of traffic which the canal might otherwise obtain cannot be accurately determined. Even with an annual average traffic of 10,000,000 tons it would about pay expenses and be of great commercial benefit to the localities which it would serve. Furthermore, the traffic between Pittsburg and the Lake Erie ports is increasing rapidly and no doubt will soon require additional transportation facilities. It is a question how far the railroads now handling this traffic can increase their capacity because of the difficulty of enlarging their terminals, even at enormous cost.

The communities interested in this canal propose to bond themselves in order to raise the money for the cost of construction, and have already obtained the legislation necessary for such bond issues. They ask that the work be carried out under the direction of the Secretary of War and the Corps of Engineers, and that the Federal Government construct the harbor in Lake Erie and make the improvements necessary in the rivers of the Pittsburg district.

In view of the great benefits which would result from the construction of this waterway, the Commission believes that the Federal Government is justified in coöperating with the localities which are to furnish the funds to the extent of building the approaches to the canal and of lending them the Army engineers to perform the engineering work necessary for its construction.

The Commission accordingly recommends that when $10,000,000 is available in cash, and bonds to the amount of $50,000,000, or as much more as is necessary in the opinion of the Secretary of War to insure its completion, have been authorized and the legality of such bonds has been certified by competent legal authority, he shall direct the Chief of Engineers to detail without charge for services such officers from the Corps of Engineers as he shall deem necessary to perform the engineering work necessary for the construction of the proposed canal.

The commission further recommends that when the work of constructing the canal has actually begun, Congress, if satisfied that it will be completed, shall appropriate the funds necessary for an adequate harbor in Indian Creek at the Lake Erie end and for the necessary improvement of the Ohio River in the Pittsburg district, the same to be completed by the time the canal shall be ready for operation, but the Government should not be required to purchase any land in making these improvements.

The plan of coöperation here proposed is intended to leave the canal essentially a local enterprise, and the above recommendations for the coöperation of the Federal Government in constructing this waterway, are not to be construed as committing or obligating the Government, now or at any future time, to assume any financial responsibility whatever in any way related to its construction, maintenance, or operation.

A Projected Canal from Lake Erie to Lake Michigan

A report on this project has been made by Col. George J. Zinn, Corps of Engineers, and printed by the Commission as Document No. 21. The proposed route of this canal is from Toledo up the Maumee River as far as Fort Wayne, and from this point to Lake Michigan by an artificial waterway. Nothing more than a preliminary reconnoissance of the route to develop its engineering difficulties, cost, etc., has been made.

Concerning the capacity of the proposed canal, there have been two propositions, one to construct a ship canal with depth of 24 ft., the other a barge canal with not over 16 ft. depth. The Commission advises that the 24-ft. canal should not be considered. Not only would the cost of construction be great and the difficulty of providing adequate water supply for the summit level be serious, but if the canal were constructed the Commission believes that large lake steamers would prefer the longer lake route by way of the Straits of Mackinaw in preference to the slow and risky passage through the proposed canal.

The Commission concludes that if the canal could obtain any considerable amount of traffic, it would be through its furnishing a connection with the enlarged Erie Canal now under construction by the State of New York. It states that a type of barge of about 2000 tons capacity has been especially designed to pass through the new Erie Canal and at the same time be safe to navigate the lakes. By giving the proposed canal the same depth and width as the New York Barge Canal, such barges could make the trip from the southern end of Lake Michigan through this proposed canal, then through Lake Erie to the New York Barge Canal and so on to tidewater. The Commission concludes, therefore, that 14 ft. depth would be sufficient for this waterway.

After an extended discussion of the possibilities of traffic on the proposed canal, the Commission concludes that it is worth while to have a survey made by the Corps of Engineers to determine the most feasible route, the probable cost of construction and the traffic possibilities. An appropriation of not more than $15,000 is recommended for this purpose, which it is evident would be sufficient for nothing more than a rough preliminary survey.

Proposed Anacostia and Chesapeake Canal

The Commission was authorized to investigate this old project, originally promoted in 1828. The proposed canal would extend from the Anacostia River, near the District of Columbia boundary line, to some point on Chesapeake Bay. The purpose of the proposed canal would be to shorten the water route between Washington and Baltimore. The present distance is 210 miles, which would be reduced to 79 miles by the proposed canal. A sea-level canal, however, would require a cut 180 ft. to 200 ft., which would be enormously expensive. There is no adequate water supply for a lock canal of any size, and even if there were, the slow speed in passing through a canal of such a length would make vessels prefer the longer distance through the open river and bay.

Legislation to Compel Co-operation between Railways and Waterways

The Commission gives much attention to the need for protecting water transportation from unfair competition by railway lines. Amendments to the Interstate Commerce law made in 1910 prohibit railways which have lowered their rates to compete with waterways from raising them again after the waterway competition has been removed. The Commission believes, however, that further legislation is necessary and it favors giving power to the Interstate Commerce Commission to control joint rates and through routes between rail and water lines, so that it may compel such routes and such rates to be established whenever the public interest requires. It also favors giving power to the Interstate Commerce Commission to compel physical connections between railways and waterways wherever through routes are necessary.

The Commission refers to the fact that the railways through their ownership of steamships on the Great Lakes dominate the lake and rail package-freight business, and also to a considerable extent the grain traffic. Railway lines also dominate transportation on Long Island Sound and to a considerable extent the coastwise trade. The Commission recommends that the Interstate Commerce Commission be given jurisdiction over all lines of water transportation which are under railroad control, and also over all independent water carriers which operate over a specified route with regular schedules.

Public Control of Water Terminals

The most essential requirement, says the Commission, for the preservation of water transportation is the establishment of adequate terminals. Private corporations and railway companies control the water terminals in nearly all the large cities of the country, not only at the sea-

ports, but on the Great Lakes and the principal rivers. The public control of waterway terminals, however, says the Commission, is rather a local or state problem than a federal problem. The Commission recommends that the federal government shall not make further improvements in rivers and harbors unless the localities affected give assurance that proper water terminals and other necessary adjuncts to navigation shall be furnished by municipal or private enterprise and at reasonable charges. In many cases it is not the condition of the channels so much as it is the lack of terminals that retards the development of water transportation. With water terminals in private control, expenditure by the federal government to reduce the cost of water transportation may benefit not the public but the interests which control the terminals.

Are Storage Reservoirs Practicable for Preventing Floods and Aiding Navigation?

The Commission devotes some ten pages of its report to a discussion of the practicability of storage reservoirs. The conclusions reached may be summarized as follows:

(1) With the development of the country, the necessity on many streams of some means of flood-protection prevention has become most urgent.

(2) Flood prevention by means of storage reservoirs is most practicable where the value of property which may be damaged by flood is greatest and where the reservoirs can be also used for power development and aiding navigation.

(3) The information available is not sufficient to enable the Commission to specify on what streams reservoir construction would yield results commensurate with the cost. Each case must be considered on its merits, and the Commission urges the necessity of careful studies such as those made by the Pittsburg Flood Commission.

(4) The federal government has no constitutional authority to engage in works intended primarily for flood prevention or power development. The Commission holds that flood prevention is primarily a local problem and that the federal government can only participate in the work where a substantial improvement to navigation will result.

(5) The extent to which the federal government may share the expense of constructing storage reservoirs on the head waters of a navigable stream must be determined in each case by government experts.

(6) The Commission believes that the federal government is justified in investigating the proposed reservoirs on the Allegheny and Monongahela head waters. The Commission recommends, therefore, that a board of engineers be created to investigate the question whether the proposed reservoirs planned by the Pittsburg Flood Commission are needed to furnish sufficient water supply for operating the government's locks and dams on the Ohio River and its tributaries and to what extent the benefits of navigation would justify the federal government in sharing the expense.

Influence of Forests upon Navigation and Flood Prevention

The Commission devotes ten pages to a review of the conflicting opinions, statements and reports upon this subject. This review deserves high commendation for its fair and judicial character. The Commission says that the officers of the Corps of Engineers and meteorologists are, as a rule, inclined to minimize the influence of forests upon rainfall and stream flow, while geologists, foresters and others are inclined to emphasize it and civilian engineers are about equally divided.

The Commission reviews the investigations made by Prof. Mead, of the University of Wisconsin, and those of Colonel Burr, of the Corps of Engineers, both of which showed that no particular variation in stream flow could be traced to the large changes in forest cover which have taken place in certain drainage basins. It also reviews the studies made by M. O. Leighton, of the Geological Survey, and Messrs. Hall and Maxwell, of the Forest Service, tending to show that a number of streams in the eastern United States were becoming more irregular in their flow. The Commission rejects all these records, however, as not finally settling the question.

The final conclusion reached by the Commission is that whatever influence forests may exert upon precipitation, run-off and erosion, it will evidently be greatest in mountainous regions. In no case, however, can forests be relied upon to prevent either floods or low-water conditions. There is substantial agreement of all the witnesses on this point.

The prevention of erosion undoubtedly outweighs all other benefits of forestation, and the Commission favors the prevention of forest removal on mountain slopes wherever the land is unsuitable for agricultural purposes. It urges the reforestation of tracts which have already been stripped of timber, not only when located at the headwaters of navigable streams, but wherever this would be the most valuable use of the land.

Of at least equal importance with forest preservation are the prevention of forest fires, the regulation of hillside farming, and prohibition of stripping the forest cover on mountain sides where the soil cover is thin. The Commission concludes that the chief responsibility for forest preservation and protection of lands from erosion rests with the separate states, rather than with the federal government.

Water Power Development and Control

The final section of the report, including nearly one-third of its contents, is devoted to the problem of water-power development and control and the relation thereto of the federal government. The Commission points out that under present conditions and present laws there can be no extensive water-power development within the public domain. Any such development must depend on permits which the law permits the Secretary of Agriculture and the Secretary of the Interior to sign. These permits are revocable at will, are unassignable, unmortgageable and subject to interference by the location of mining claims and the transfer of land from one jurisdiction to another. The conditions respecting the construction of water-power plants on navigable streams are not much better.

For the conservation of the nation's fuel supply, it is urgently desirable that water power now undeveloped should be put to work. Discussing the relative cost of steam power and water power, the Commission suggests that an average saving in cost of water power over the cost of power produced by steam engines would be $20 per hp., per annum. At this rate, allowing a return of 10% per annum on the investment, water power can be developed up to a capital cost of $200 per hp. in excess of the cost of a steam plant.

Reference is made to the recent report of Herbert Knox Smith, Commissioner of Corporations, on the concentration of water-power ownership; and the Commission points out that this subject is important, not so much that financiers and promoters find monopoly advantageous as that economic considerations and the natural character of the business make monopoly almost inevitable and perhaps desirable when subject to public regulation.

Turning then to the legal aspect of water-power control, the Commission reviews at great length and with much ability the varying laws and customs in different states and different sections with respect to riparian rights. The Commission thoroughly establishes the fact that the federal government has supreme authority not only over navigable streams, but over their tributaries, so that the authority of Congress reaches to the source of every stream. Constructions of any sort in or over streams, either navigable or non-navigable, are therefore within the control of Congress. Not only has the federal government authority to control water-power plants because of their relation to streams used for navigation, but it also has authority because a large part of the business in connection with the production and sale of hydro-

electric power extends across state boundaries and has been decided to be interstate commerce.

Having established the legal and constitutional right of the federal government to control water-power development, the Commission proceeds to special recommendations, taking up first the general dam act passed June 23, 1910. It declares that experience has shown that the limited-term franchise or grant, provided in this law, does not encourage water-power development or protect the public interest. Under the terms of this act, the franchise or grant is limited to 50 years, and no provision whatever is made for the disposition of the property at the expiration of the grant. If the property is to be turned over to the government at the end of the term, which is a fair supposition, then the business must earn enough profit to amortize the investment during this period. The company must, therefore, be permitted to charge rates high enough to provide for this amortization; and if prevented from doing so by the regulations of some state commission or by outside competition, the result will be a loss to the investors.

The Commission, therefore, recommends that a more equitable form of franchise should be granted to those seeking to develop water power. Referring to the three ordinary forms of franchise, namely, the grant in perpetuity, the indeterminate grant and the limited-term grant, the Commission says that the grant in perpetuity, which is generally demanded by water-power promoters, may be dismissed at once as not consistent with the public interests. The grant for an indeterminate term has proved successful with municipal utilities, but the Commission doubts its applicability to water-power grants widely scattered. The Commission, therefore, favors some form of limited-term grant, but specifies that the grant should contain provisions which will insure the actual investor of the permanency and security of his investment. Any such grant should include provision for compensation to the holder of the grant at the end of the term if the grant is not renewed. This compensation should extend merely to the actual physical value of the property taken over by the government.

Concerning the development of water power in the public domain, the Commission recommends amendments to the existing laws. It suggests first that the mining laws be so altered that mining entries can be made in localities suitable to water-power development only under conditions which will not interfere with such water power. It also recommends that homestead and other public-land entries should be subject to the right of the federal government to grant rights-of-way across the land for electric transmission lines or other adjuncts to hydro-electric plants. The Commission suggests the advisability of creating some executive board which shall have authority under suitable conditions to grant permits for water-power development and transmission, either upon the public domain or on navigable streams. Congress would then make laws establishing a general policy under which such grants would be made, but would not as at present pass a special act for each grant.

Concerning compensation for grants for water-power development, the Commission declares that as this constitutes a special privilege, the government is surely entitled to proper compensation. Such compensation, however, it believes, should not be made a means of raising revenue for general purposes, but should be so fixed as to compensate the government for the necessary cost of surveys, inspection, etc., and to control the use of streams in the interest of the public.

It will be apparent from the above review that the Commission takes strong and emphatic ground in favor of federal control of water-power development. In a concluding paragraph of the report the Commission says that while in its opinion the business has not yet reached such a stage of development as to make federal control now desirable, it is rapidly approaching that stage. The report concludes as follows:

> The Commission would recommend that for the present all questions arising between the water power company and the consumers of its product or service be left to the control of the several States within which such companies may be doing business, except in cases where the authority and power of neither municipalities or the State are adequate to protect the public interest; but with the fullest understanding that within the comparatively near future this feature of its regulation must also inevitably pass to the federal government.

Great Possibilities in Gas and Mining engineering may be indicated by the titles of the two following papers taken from the list of those to be presented before the agricultural chemistry section of the Eighth International Congress of Applied Chemistry: "The Combustible Gases Excreted by Cattle" and "The Mineral Constituents of the New Born Calf."

Backwater Wave in the Spillway Channel, Gatun, Panama Canal

By Edward C. Sherman*

The spillway section of the Gatun Dam will be a structure of ogee section, curved in plan and situated about midway between the ends of the earth dam at a point where a suitable rock foundation was found. In order to permit closure by the earth dam of the river channel and the canal and diversion channels which the French company had excavated, an outlet for the Chagres River was provided by making a cut about 300 ft. wide through the rock at the site of the spillway. The concrete floor and side walls of this cut were placed, so far as was practicable prior to the construction of the dam itself, so as to protect the rock from erosion and to obtain a smooth channel for the water (Fig. 1). The spillway dam could not, of course, be constructed until work on the earth dam and the locks had advanced far enough so that backing up the lake against them would do no harm.

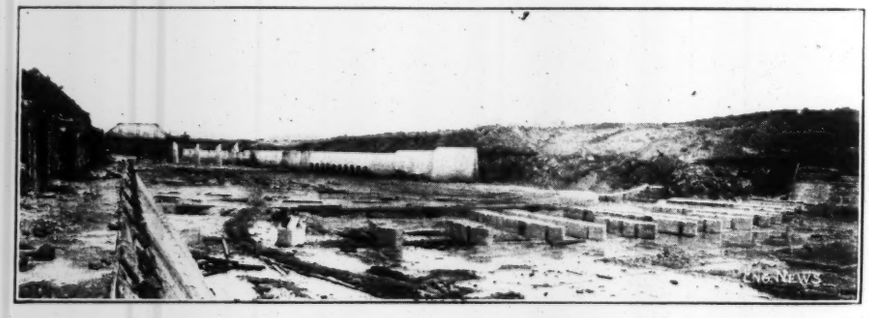

Fig. 1. Gatun Spillway. April 24, 1910

(Looking north from west wall; foundations for valve and coffer-dam piers in foreground. Bridge and piers in background.)

As soon as the channel was lined and arrangements made for the subsequent construction of the dam, the former outlets were closed and the river began to flow through the new artificial channel. Trains carrying spoil from Culebra Cut for use in that part of the earth dam lying west of the spillway crossed on a timber trestle some distance above the site of the spillway dam. There was danger that this trestle might be carried away by a great freshet such as not infrequently comes down the Chagres in the rainy season and, realizing this, the engineer of the Atlantic Division suggested a more stable structure.

The bridge which he proposed and which was subsequently built, was constructed largely of old steel spans from the Panama R.R.; the piers, five in number, being spaced to accommodate the superstructures which were available,

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and although referred to as a "permanent" bridge as noted above, it was decided that it would be retained only in case it was found in practice that the piers offered no serious obstruction to the current.

The appearance of a backwater wave just above the bridge piers and extending entirely across the channel was noted before the lake had risen very high and when, in the freshet which occurred early in December, 1910, the lake rose

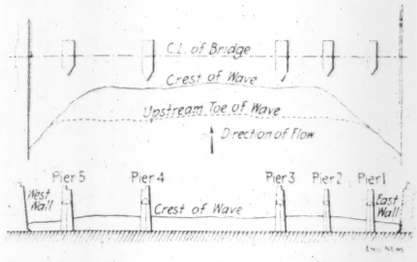

FIG. 2. WAVE AT BRIDGE OVER GATUN SPILLWAY

(Lake at El. 21.3.)

to elevation 26, giving a depth of 16 ft. at the upper end of the channel, the wave reached a rather alarming size. The question naturally arose as to what would happen in the event of a much larger flood and especially what would happen after the completion of the dam, when, with all crest gates open, some 140,000 cu.ft. per sec. would be poured down the channel. The writer was instructed to investigate and report on the matter with special reference to the height of the backwater wave, the probability of the channel walls being overtopped and the danger of the bridge being damaged or wrecked.

Approximate measurements of the height of the wave with the lake at about elevations 20 and 26, had already been taken and the channel had been rated by current-meter measurements so that the corresponding discharges were known. The substitution of the known quantities in theoretical formulas indicated that in this case they could not be used with any confidence in determining the probable wave height for other lake levels. It seemed best, therefore, to have more complete observations taken at a number of different lake levels while the lake was falling after the freshet so that suitable coefficients for use in the formula or perhaps an empirical formula for the particular case under consideration might be derived. The Third Division of the Chief Engineer's Office, which was charged, among other duties, with making all stream gagings, was asked to take such observations, and as the lake fell to its normal level observations sufficiently complete for use were obtained for four different lake levels—at elevations 26, 22.4, 21.3 and 21.0.

With only four satisfactory observations of wave height, three of which were close together, and all subject to the error due to considering only one longitudinal section of a stream 285 ft. wide, it did not seem advisable to attempt to derive a purely empirical formula. Accordingly a rational theoretical form was taken and the constants manipulated until it fitted all the observations very closely. The resulting equation was as follows:

$$Q = 797\ (D + H)^{\frac{3}{2}} - 174\ H^{\frac{5}{2}}$$

in which

Q = Cu.ft. per sec. flow in channel,
D = Depth of the undisturbed stream,
H = Height of wave, i.e., the additional depth caused by the obstructions.

Knowing approximately what the depth D in the channel would be with a discharge of 140,000 cu.ft. per sec., made it possible to determine from the formula the probable height of the wave under the given conditions. The general form of the backwater wave was as shown in Fig. 2, and longitudinal profiles of the four observed waves and of the one predicted for a discharge of 140,000 sec.ft. are shown on a somewhat exaggerated scale in Fig. 3. It is seen that the total depth at the wave will probably be about 33.5 ft., and that the side walls may be overtopped whenever the discharge is in excess of about 80,000 cu.ft. per sec.

As the bottom chords of the bridge girders adjacent to the middle truss span are about 33.5 ft. above the channel floor, it is evident that they would be likely to suffer serious damage from drift should the assumed discharge take place. Believing that the probability of a freshet of such magnitude was small, and that when such an amount is discharged over the completed dam there will be little or no drift on account of the large size and irregular outline of the lake, it was recommended that the bridge be left in place and the chance of damage be taken until the completion of the work when, if it appears desirable to have a permanent bridge across the spillway channel, the present structure should be removed and a new one, without channel piers, built in its stead. With the piers removed there will be no danger of the walls being overtopped.

In order that overtopping the walls

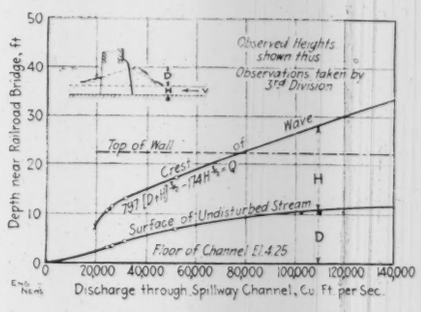

FIG. 4. PLOT OF FORMULA FOR WAVE CAUSED BY PIER OBSTRUCTURES, GATUN SPILLWAY CHANNEL

while the obstructions remain might not cause serious damage, it was recommended that certain temporary protective paving be placed on the banks for a short distance each side of the bridge.

No large freshets have occurred since the investigations were made, and the work on the dam is now advanced to a point where, in any event, only a comparatively small flow can be passed, Gatun Lake serving to hold the surplus until the flood subsides. It would appear therefore that the danger has been passed and the conclusions justified.

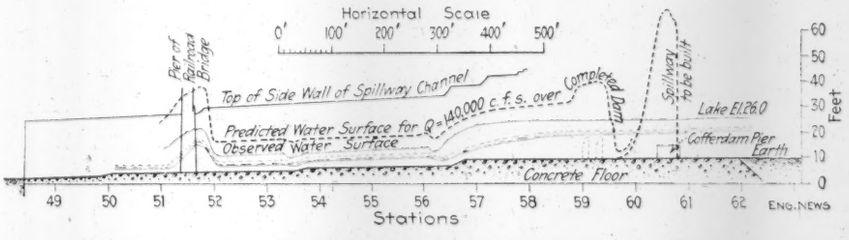

FIG. 3. PROFILES OF STREAM THROUGH GATUN SPILLWAY CHANNEL

The Effect of Seawater on Concrete is succinctly stated at the end of a paper entitled "Marine or Iron-Ore Cements," presented by H. E. Brown to the annual convention of the National Association of Cement Users, Kansas City, Mar. 11-16, 1912. The summary is as follows:

When disintegrating effects occur in concrete exposed to the action of sea water, and where there are variations in level of the water, due to wind, wave and tide, these disintegrating effects may be rightly divided into two main causes—mechanical and chemical. Mechanical action is influenced (1) by alternate drying and wetting of the surfaces, due to wind, wave and tidal action; (2) abrasions from ice or floating objects; (3) formation of expansive crystals where freezing occurs, or where double salts may be formed within the concrete body.

This brings us to the second main cause of disintegration, namely, chemical action, since the formation of the double salts is first brought about by substitutive chemical reactions. Such double salts as calcium aluminum sulphate occupy more space than the original compounds. The dissolving, replacing and formation of these new crystalline substances act in the same manner as ice crystals within the rocks. Such ice crystals have been very effective in producing disintegration of the archaian rocks covering the earth's surface.

The hydraulic cements which are best fitted to withstand satisfactorily all disintegrating effects of sea water (excepting those which are purely mechanical) are those which give the greatest density of concrete structure, and which are relatively high in silica, low in magnesia and sulphur trioxide, and in which the content of alumina does not exceed that of the ferric oxide.

Benjamin Guggenheim

The sinking of the ill-fated steamship "Titanic" carried to their graves many well known men prominent in the public life and the industrial development of America. Among these was Benjamin Guggenheim, one of the younger of the seven sons of the late Meyer Guggenheim, who was one of the most remarkable characters of the past generation, and the founder of an immense fortune.

Benjamin Guggenheim was born in 1865. At 20 years of age he was sent by his father to Leadville, Colo., to take charge of extensive mining interests. Here he studied not only the commercial but the engineering and metallurgical side of the mining industry and was among the first to realize the great possibilities in the smelting industry.

The first smelting plant at Pueblo, Colo., was built under his immediate direction. In all the western mining operations of the family, Benjamin was a leading spirit.

BENJAMIN GUGGENHEIM

In 1903 he entered into the manufacture of mining machinery, and with his great wealth and business sagacity built up the plant of the Power & Mining Machinery Co., at Cudahy, Wis., a suburb of Milwaukee. Three years later this company was merged with the International Steam Pump Co., of which Mr. Guggenheim ultimately became president.

Under his direction the business has been vastly extended, and includes not only numerous plants in the United States but others in England and France. Although not an engineer, Mr. Guggenheim had sufficient breadth of view to appreciate engineering as well as industrial and commercial problems, and no small part of his success in mining and manufacturing was a just appreciation of the service rendered by his many engineering associates.

Wind Storms in the South and West, Apr. 20-22, caused a large property damage and considerable loss of life. On Apr. 20 a storm swept in a northeasterly direction through Oklahoma and Logan Counties, in Oklahoma State, and on the same date a tornado struck the town of Bison, in Rush County, Kansas. A number of persons were reported killed or injured at Bison and many buildings were blown down, including a grain elevator. In Oklahoma, a total of about 100 buildings, including a stone business structure, were completely wrecked.

On Apr. 21 a severe wind storm swept across parts of Illinois and Indiana. Reports state that a total of 34 persons were killed in these states. The heaviest damage was done in the vicinity of Bush, Ill., where 18 persons were killed and 100 others injured. At this place the buildings of the Western Coal & Mining Co., the post office, general store and hotel were destroyed, together with 40 dwellings. Reports state that 35 other dwellings were partly wrecked.

On Apr. 22 tornadoes were reported in western and middle Georgia and in eastern Alabama. The principal towns affected in Alabama were Adamsville, Pinckney City and Brookside. In these three places 15 persons were reported killed. In Georgia, six persons were reported killed at Newborn, in Newborn County, and many others injured. Two persons were reported killed at Hampton, Ga. Other towns affected in this state were Bowden, Cedartown and Fayetteville.

The number of persons killed by the wind storms on these three days in the different sections of the country totaled about 100.

The Mississippi River Flood is passing down the river with great accompanying damage and loss of life. On Apr. 23, the U. S. Weather Bureau reports it falling steadily between the mouth of the Ohio and Memphis, slowly below that point as far south as Natchez and about stationary below that point. The rise in the lower river is retarded on account of numerous breaks in levees. A further rise is probable below the mouth of the Red River when overflow waters return to the main channel. The river stages on Apr. 23 were as follows: Cairo, 44.8; Memphis, 39.9; Helena, 54.2, Arkansas City, 52.9; Greenville, 46.8; Vicksburg, 48.4; Natchez, 50.1; Donaldsonville, 32.2, and New Orleans, 20.2. Numerous breaks in the lower river levees occurred from Apr. 13 to 17, and over 2000 square miles in Arkansas and Louisiana are reported flooded. Congress appropriated an additional $300,000 for protection and relief on Apr. 16, and many of the larger cities have organized relief expeditions which are now cruising about the flooded territory.

The Gate Caisson for the new dry dock at Pearl Harbor, near Honolulu, Hawaii, was launched at the Union Iron Works, San Francisco, a week ago. This caisson is 126 ft. long, 20 ft. wide and 43½ ft. high over frames. It weighed nearly 1000 tons at launching. It is to be towed to Hawaii by a tug. A large amount of concrete is in the caisson to give it stability and to sink it nearly to the proper level. The further manipulation is done by water ballast.

The St. Louis Terminal Association is a conspiracy in restraint of trade by decision of the U. S. Supreme Court, under an opinion written by Judge Lurton, who also wrote the recent opinion greatly broadening the scope of patent monopolies. The Terminal Railroad Association originally owned the Eads bridge only, but later it bought the Merchants' Bridge (which had been built to establish competition in railway river crossings at St. Louis) and the Wiggins Ferry Co. By this means the Association, owned by 14 of the railways entering St. Louis, had full control over all railway traffic entering St. Louis from the East, and for years past it has made a separate charge on all freight and all passengers crossing the river, alleging that this is a necessary and proper charge for the terminal handling and distribution. The Association has developed an extensive network of terminal tracks in and around St. Louis, and has transacted the terminal business for all railways. But since only part of the railways at St. Louis are members of the Association, the others being kept out deliberately, the Supreme Court finds that the intent of the Association is against the Sherman Law. The Association is, therefore, ordered to reorganize and to admit into its membership all railways that must use its terminal facilities. It is ordered also to change certain of its business practices, the chief one being its requirement that all freight handled by it must be billed to East St. Louis and rebilled from there, giving opportunity for the terminal charge. This terminal charge, locally called the Bridge Arbitrary, has long been a matter of bitter criticism by all St. Louis interests, and the persistence of the railways through the Terminal Association in maintaining this charge was the moving force in the project for the Municipal Bridge (therefore called Free Bridge), which is now under construction and nearly completed.

Threatened Strike Delayed—The locomotive engineers on the railways east of the Mississippi and north of the Ohio, who recently presented demands for increase of pay, were refused the increase by the Conference Committee of Managers of the railways on Apr. 22. Action began at once to start a strike on Apr. 24, but an immediate offer on the part of government authorities to mediate delayed this action. The offer was made by M. A. Knapp, presiding judge of the U. S. Commerce Court, and C. P. Neill, U. S. Commissioner of Labor. The Erdman Act provides for arbitration when request therefor is made to the government by the parties to an industrial dispute, but in this instance the government representatives most directly concerned made the initial offer.

The pay scale asked for by the locomotive engineers fixes $4.40 to $4.60 for passenger engines, according to size, and $5.25 to $5.75 (and $7 for Mallets) for freight engines, these rates being per 100 miles or less with pro rata pay for greater distances, and in the case of freight engines also for 10 hours or less. Various accessory provisions make up the rest of the demand. The railway managers claim that the demands represent a virtual increase of 18% in pay, and that the railways cannot afford this.

A Three Mile Tunnel in Montreal is about to be built by Mackenzie, Mann & Co. for the Canadian Northern Montreal Tunnel & Terminal Co. The tunnel, double-track, will pass through Mt. Royal, and will give the Canadian Northern Ry. entrance from the west into the heart of Montreal. The terminal station of the railway will be located in the center of the city, between Cathcart and Lagauchetiere St. and between Mansfield

and St. Monique St. It is to be below grade, and will have about 12 station tracks, half of them through tracks and the others stub terminal tracks, with station platforms about 1000 ft. long. The through tracks will continue from the station on viaduct to the bank of the St. Lawrence River, connecting there with the tracks of the Harbor Commissioners and also leading to an elevated freight yard. The Harbor Commissioners' tracks will give a through connection to the East. An important auxiliary of the tunnel and terminal project is the development of a new residential suburb at the back of Mt. Royal. When the tunnel and station are completed this new section will be within six to ten minutes of the center of the city. Surveys for the entire project are now under way and plans are being made, under direction of Mr. S. P. Brown, as Chief Engineer. It is expected that actual construction work will be started some time in May.

An $8,800,000 Bond Issue was recently voted at San Francisco., Calif., for a new city hall and for additional land for a civic center.

Permission to Withdraw the Lowest bid on account of an alleged error was refused by the Board of Public Works of San Francisco a few days ago. Bates, Borland & Ayer, of Oakland, bid $225,-025 on work for the completion of the Geary Street Municipal Railway, from Kearney St. to Fifth Ave. This was $28,-000 below the lowest bidder and $50,000 below the estimate of the city engineer. Mayor Rolph and the Board named held that no proof of error was before them and the contractor notified the mayor that he would proceed with the work.

Concrete Buildings are not the only ones which are affected by freezing weather during construction, according to the following news note in the St. Louis "Globe-Democrat" of Apr. 1:

A two-story brick building at Alcott and Lilian avenues, Strattman Heights, collapsed early yesterday morning. The building was nearing completion. The loss is estimated at $5000.

The collapse of the building was caused by mortar which was mixed when the temperature was 16° below zero being used in construction, it is said.

Bids for a Portion of the Passaic Valley trunk sewer are to be received on May 14, by the Passaic Valley Sewerage Commissioners, at 29 Clinton St., Newark, N. J. Section 15, as this portion of the sewer is called, is located in the city of Passaic and the township of Acquackanonk. The section has a total length of 7275 ft. Of this 3445 ft. will be a 90-in. concrete sewer in a trench ranging from 15 ft. to 35 ft. in depth. There will be 1080 ft. of 90-in. concrete sewer in tunnel and 2750 ft. of 81-in. concrete sewer in tunnels. Altogether there will be 10,700 cu.yd. of portland cement concrete, partly in trench and partly in tunnel, and also 1500 cu.yd. of rock excavation in trench and 200 cu.yd. of brick masonry in manholes. William M. Brown is Chief Engineer and John S. Gibson is Clerk of the Commission named.

Personals

Mr. J. A. S. Redfield, Division Engineer of the Chicago & Northwestern Ry., has been transferred from Sioux City, Iowa, to North Fond du Lac, Wis.

Mr. F. W. Taylor, Master Mechanic of the Illinois Central R.R. at Mattoon, Ill., has been transferred to Waterloo, Iowa, succeeding Mr. J. A. Bell, who is transferred to Mattoon.

Mr. John P. Congdon, M. Am. Soc. C. E., formerly Supervising Engineer of the Oregon Short Line R.R., has opened offices in the Empire Bldg., Boise, Idaho, as a consulting engineer.

Mr. T. C. Hu, a graduate in civil engineering of Cornell University, class of 1905, has been made Dean of the Civil Engineering Department of the Nan Yang University at Shanghai, China.

Mr. Richard L. Humphrey, M. Am. Soc. C. E., of Philadelphia, Penn., President of the National Association of Cement Users, has been elected a member of the Institution of Civil Engineers of Great Britain.

Mr. Chas. Waldrich, for many years connected with the engineering departments of the Pennsylvania R.R. and Missouri Pacific Ry., is now associated with the Miller Engineering Co., of Little Rock, Ark., engineers and contractors.

Mr. C. H. Fox, Resident Engineer of the Canadian Pacific Ry. at Fort William, Ont., has been promoted to be Assistant Engineer of the Manitoba division with headquarters at Winnipeg, Man., succeeding Mr. J. C. Holden, promoted.

Mr. J. C. Holden, Assoc. M. Can. Soc. C. E., Assistant Engineer of the Manitoba division of the Canadian Pacific Ry., at Winnipeg, Man., has been promoted to be Division Engineer of the same division, succeeding Mr. F. Lee, promoted.

Mr. Joseph P. Sherer, for five years a member of the Board of Public Works of Milwaukee, Wis., before the board was replaced by a single commissioner, has been appointed Deputy Commissioner of Public Works at a salary of $2200 per annum.

Mr. Chas. T. Waring, Assoc. M. Am. Soc. C. E., formerly Division Engineer of the Cape Cod Canal, has been appointed Resident Engineer, succeeding Mr. H. W. Durham, M. Am. Soc. C. E., whose resignation was recently noted in these columns.

Mr. H. H. Adams, formerly General Manager of the Toronto, Hamilton & Buffalo R.R., at Hamilton, Ont., has been elected President of the Kansas City Terminal Ry., at Kansas City, Mo., to succeed Mr. W. S. Kinnear, M. Am. Soc. C. E., resigned.

Mr. William O. Galbreath, Assoc. M. Am. Soc. C. E., formerly Division Engineer of the National Rys. of Mexico, at Chihuahua, Mex., has been appointed Engineer of Maintenance-of-Way of the Missouri & North Arkansas R.R., with headquarters at Eureka Springs, Ark.

Dr. E. W. Lazell, formerly of the Spackmann Engineering Co., of Philadelphia, Penn., and more recently Consulting Engineer of the Charles Warner Co., of Wilmington, Del., has become a member of the firm of Edwards & Lazell, Consulting and Chemical Engineers, of Portland, Ore.

Messrs. W. W. Gaffin, for a number of years Division Engineer of the Chicago & Northwestern Ry., at North Fond du Lac, Wis., and F. Gehri, of Janesville, Wis., have formed a partnership under the firm name of Gaffin & Gehri, for the general practice of engineering, with offices at Fond du Lac.

Mr. John Callaghan, M. Am. Soc. C. E., formerly Division Engineer of the Grand Trunk Pacific Ry., at Fitzhugh, Alta., has been made Chief Engineer of the Pacific Great Eastern Ry., in British Columbia. Mr. Callaghan was formerly in the construction department of the Canadian Pacific Ry., at Winnipeg, Man.

Mr. Steven P. Brown, M. Am. Soc. C. E., former Chief Engineer of the Tidewater Building Co. & T. B. Bryson, contractors for a portion of the Fourth Ave. subway, Brooklyn, N. Y., is now Managing Engineer for Mackenzie, Mann & Co. and Chief Engineer of the Canadian Northern Montreal Tunnel & Terminal Co., of Montreal, Que.

Mr. Edwin C. Blanchard, General Superintendent of the Northern Pacific Ry., Lines east of Paradise, has been promoted to be Fourth Vice-President and General Manager of the lines west of Paradise, succeeding Mr. H. C. Nutt, whose resignation was noted in our issue of last week. Mr. Blanchard began railway work in 1873 at the age of 19 years as an agent and telegraph operator on the Burlington, Cedar Rapids & Minnesota R.R. He entered the service of the Northern Pacific Ry. in 1883 as a train dispatcher and has steadily advanced through the positions of Chief Train Dispatcher, Division Superintendent and General Superintendent.

Mr. William Wainwright, Vice-President of the Grand Trunk Ry. System, has been chosen Temporary President of the Grand Trunk Ry., succeeding the late Charles M. Hays. Mr. E. J. Chamberlain, Vice-President and General Manager of the Grand Trunk Pacific Ry., has been chosen Temporary President succeeding Mr. Hays. Mr. Wainwright is an Englishman and served four years as a clerk on the Manchester, Sheffield & Lincolnshire Ry. before coming to Canada in 1862. Since then he has been continuously with the Grand Trunk Ry. Mr. Chamberlain is a native of New Hampshire and was formerly General Manager of the Canada Atlantic Ry. (now a part of the Grand Trunk System).

Obituary

Noyes F. Palmer, a civil engineer and city surveyor of Brooklyn, N. Y., died on Apr. 18, aged 66 years.

Washington A. Roebling, 2nd, grandson of Washington A. Roebling, who completed Brooklyn Bridge after the death of John A. Roebling, its original engineer, was among the lost passengers on the "Titanic" which sank on Apr. 15.

Edgar J. Meyer, a graduate in mechanical engineering of Cornell University, class of 1905, was lost in the sinking of the "Titanic" on Apr. 15. He was 28 years old and had been married about two years. He had been in business with his brother in New York City.

James P. Coleman, for several years on the engineering staff of the Illinois Central R.R., and recently with the MacArthur Bros., Contractors, of Chicago, Ill., died at the Lakeside Hospital, Chicago, on Apr. 13. He was about 63 years old and had been a member of the Western Society of Engineers for 22 years.

Dr. Paul C. Freer, Director of the United States Government Bureau of Science in the Philippine Islands, died in Manila on Apr. 17, aged 50 years. For 14 years, 1890 to 1904, he was Professor of Chemistry at the University of Michigan. He had been in his present position since 1905. He was Editor of the "Philippine Journal of Science," a

member of the American Chemical Society and other chemical and medical associations of the United States and Europe.

John B. Thayer, Second Vice-President of the Pennsylvania R.R., was drowned in the sinking of the "Titanic" on Apr. 15. Mr. Thayer was returning from an extended leave of absence in Europe in search of rest and health after many years of busy life as a railway executive. He was accompanied by Mrs. Thayer and his 17-year old son, both of whom were rescued. Mr. Thayer was born in Philadelphia, Penn., in 1862. He attended the University of Pennsylvania for a short time, but at 19 years of age left school to become a clerk in the auditor's office of the Empire Fast Freight Line. Two years later, in 1883, he became a clerk in the freight department of the Pennsylvana R.R., and ever since that time had been continuously in the service of this railway. He rose through various clerical positions in the freight department to be General Freight Agent in 1897. In 1903 he was elected Fifth Vice-President, just 20 years from his start with the Pennsylvania R.R. as a freight clerk.

William A. Lathrop, M. Am. Inst. M. E., President of the Lehigh Coal & Navigation Co., and of the Lehigh & New England R.R., died on Apr. 12, at the City Hospital, Wilkes-Barre, Penn., following an operation for appendicitis. He was born in Springville, Penn., in 1854, and in 1875 graduated in civil engineering from Lehigh University. He afterward took a course in mining engineering. After a few years in minor positions in the Pennsylvania coal-mining district, he became Assistant to Robt. H. Sayre, Chief Engineer of the Lehigh Valley R.R. Later he became associated with Maj. I. A. Stearns, of Wilkes-Barre, and was for a time Manager of an iron mine in northern New Jersey. In 1881, as Superintendent and Engineer of the Southwest Virginia Coal Co., he developed the famous Pocahontas coal fields. Some years later he returned to Pennsylvania as Superintendent and General Manager of the Lehigh Valley Co. In 1902 he was elected President of the Webster Coal & Coke Co., and a few years later of the Lehigh Coal & Navigation Co. Mr. Lathrop was a member of the State Commission recently appointed by Governor Tener to devise plans for the protection of the surface of the anthracite region from damage by mine cave-ins. He was a trustee of Lehigh University and a well known citizen of Philadelphia.

Charles Melville Hays, President of the Grand Trunk Ry. System, was among the prominent men who lost their lives in the sinking of the "Titanic" on Apr. 15. Mr. Hays was returning home from a business trip to London, and was accompanied by his wife and daughter, both of whom were saved. Like Sir Thomas G. Shaughnessy, the head of the other great Canadian railway system, Mr. Hays began life as a poor American boy. He was born at Rock Island, Ill., on May 16, 1856. At 17 years of age he started his railway career as a clerk in the St. Louis office of the Atlantic & Pacific Ry. From 1877 to 1884 he was Secretary to the General Manager of the Missouri Pacific Ry., and for the year following held a similar position with the General Manager of the Wabash, St. Louis & Pacific Ry. From this position he rose rapidly to those of Assistant General Manager, and General Manager and Vice-President of the Wabash System. From 1896 to 1901 he was General Manager of the Grand Trunk Ry., leaving there to become President of the Southern Pacific Co. But the Grand Trunk owners soon found they could not do without his services and within a year he returned to become Second Vice-President and General Manager. On Jan. 1, 1910, he became President of the of the Grand Trunk System and of the Grand Trunk Pacific Ry., which placed him in the front rank of the world's great railway executives. A portrait and biographical sketch of Mr. Hays was published in our columns on Oct. 28, 1909.

Not least among the many brave men who were lost on the "Titanic" were her engineers. These included not only her regular crew, but several making the trip in the interests of the ship's designers and builders. The most distinguished of these was Thomas Andrews, M. I. Mech. E., one of the staff of designers of the "Titanic's" machinery, who was making the trip for his firm, Harland & Wolff, of Belfast. When last seen by any of the survivors, Mr. Andrews was in the engine room of the sinking ship.

Felix Benedict Herzog, M. Am. Inst. E. E., died in the Roosevelt Hospital, New York City, on Apr. 21. He was born in New York City in 1859 and graduated from Columbia University in 1881. He was an inventor of many electrical devices, including automatic switchboards, police calls, elevator signals, etc. He was also noted as a patent attorney and expert.

Engineering Societies

COMING MEETINGS

AIR BRAKE ASSOCIATION.
May 7-10. Annual convention at Richmond, Va. Secy., F. M. Nellis, 53 State St., Boston, Mass.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 14-16. Annual meeting at Chicago. Secy., F. H. Wentworth, 87 Milk St., Boston, Mass.

MASTER BOILER MAKERS ASSOCIATION.
May 14-17. Annual convention at Pittsburg, Penn. Secy., H. D. Vought, 95 Liberty St., New York.

AMERICAN RAILWAY ASSOCIATION.
May 15. Semi-annual meeting at New York City. Secy., W. F. Allen, 75 Church St., New York.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
May 16-18. Annual meeting at Pittsburg, Penn. Secy., F. E. Sanborn, Ohio State University, Columbus, Ohio.

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 22-25. Annual convention at Chicago, Ill. Secy., D. B. Sebastian, La Salle St. station, Chicago.

INTERNATIONAL NAVIGATION CONGRESS.
May 23-28. At Philadelphia, Penn. Secy., J. C. Sanford, 344 The Bourse, Philadelphia, Penn.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
May 28-31. Spring meeting at Cleveland, Ohio. Secy., C. W. Rice, 29 West 39th St., New York.

AMERICAN WATER WORKS ASSOCIATION.
June 3-8. Annual convention at Louisville, Ky. Secy., J. M. Diven, 271 River St., Troy, N. Y.

ASSOCIATION OF RAILWAY TELEGRAPH SUPERINTENDENTS.
June 4. Annual convention at New York City. Secy., P. W. Drew, Soo Line, Chicago, Ill.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 10-14. Annual meeting at Seattle, Wash. Secy., T. C. Martin, 29 West 39th St., New York.

MASTER CAR BUILDERS' ASSOCIATION.
June 12-14. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 17-19. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

CANADIAN ELECTRICAL ASSOCIATION.
June 19-21. Annual meeting at Ottawa, Ont. Secy., T. S. Young, 220 King St., West, Toronto, Ont.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 25-28. Annual convention at Boston, Mass. Secy., F. L. Hutchison, 33 West 39th St., New York.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 25-28. Annual convention at Seattle, Wash. Secy., C. W. Hunt, 220 West 57th St., New York.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 26-28. Annual meeting at Boston, Mass. Secy., H. H. Norris, Cornell University, Ithaca, N. Y.

Municipal Engineers of the City of New York—At the regular meeting held in the Engineering Societies Bldg., Apr. 24, Walter E. Spear, Department Engineer of the New York Board of Water Supply, presented a paper entitled "Progress on the City Tunnel of the Catskill Aqueduct."

Montana Society of Engineers—The 25th annual meeting was held at Anaconda, Mont., Apr. 11-13. Officers for the ensuing year were elected as follows: President, Robt. A. McArthur; Vice-Presidents, John H. Klepinger and Reno H. Sales; Secretary, Clinton H. Moore; Treasurer, Samuel Barker, Jr.; member of the board of trustees, Geo. A. Packard.

Papers were presented on "The Hebgen Dam," by H. H. Cochrane, and on "Good Roads," by Robt. D. Kneale. Mr. Cochrane gave a brief description of the design and purpose of the dam. It is on the Madison River, 15 miles from the Yellowstone National Park, and will be used for storing water for irrigation during the dry season. The water impounded will cover an area of 21 square miles.

Mr. Kneale's paper dwelt chiefly on the maintenance of roads, stating that a road put in with no provisions for maintaining it in good repair is a monument to poor judgment.

International Congress of Navigation—Already 29 nations have named their official delegates to the Twelfth International Congress of Navigation to be held at Philadelphia, Penn., May 23-28. The official list of delegates from Germany includes 20 names, among which are those of the leading German authorities on navigation problems. There will be, of course, many German engineers in addition to the government delegates. France is second in the list, with 19 official delegates, and the United States third, with 14. The other countries that have named official delegates are: Russia, The Netherlands, Italy, Austria, Hungary, Argentine, Belgium, Prussia, Canada, Chili, French Western Africa, Great Britain, Greece, Indo-China, Japan, Mexico, Monaco, Norway, Spain, Sweden, Switzerland, Persia, Portugal, Siam, Peru and Denmark. The total number of delegates thus far named is 108.